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REINFORCED CONCRETE ARCH BRIDGE FOR RUTLEDGE STREET

MADISON WISCONSIN

BY

EMIL S. BIRKENWALD

A THESIS SUBMITTED FOR
THE DEGREE OF BACHELOR OF SCIENCE
CIVIL ENGINEERING COURSE

THE UNIVERSITY OF WISCONSIN
1922

REINFORCED CONCRETE ARCH BRIDGE FOR RUTLEDGE STREET, MADISON, WIS.

CHAPTER I. OBJECT, SCOPE, AND PLAN OF THESIS

The existing arch bridge across the Yahara River at Rutledge Street in Madison, is so constructed that the driver of a vehicle does not have a good view of the street ahead. The object of this thesis is to design a reinforced concrete arch bridge for this location that will eliminate the present dangerous condition.

In the thesis, investigation will be confined to a graphical analysis of the arch ring and the abutments. All architectural details will be considered as being outside the scope of the thesis.

The first step in the thesis investigation was a location survey, in which the dimensions of the existing bridge, the road profile, and the topography about the bridge was secured. The question of the kind of traffic to which the bridge was subjected was solved by a traffic census.

It was decided that the dangerous condition mentioned above could be solved by decreasing the rise of the arch. The Williamson Street bridge, two blocks away, had a clearance one foot smaller than that of the existing bridge. Since it is reasonable to believe that the smallest clearance of a bridge would limit the size of craft, and since a smaller clearance would decrease the rise of the arch, the clearance of the Williamson Street bridge was adopted.

CHAPTER II. GENERAL DATA

Method of Analysis - Elastic Theory - Turneaure & Maurer's Principles of Reinforced Concrete Construction

Span --	50' 0"
Rise --	7' 6"
Inside width of bridge-	42' 0"
Width of Roadway --	32' 0"
Width of Side Walks --	5' 0"
Depth of Filling Over Crown	1' 0"
Bearing Power of Soil –	7000 lbs/sq.ft.

Dead Loads

Weight of the Pavement	150 lbs/cu.ft.
Weight of Earth Fill	120 lbs/cu.ft.
Weight of Concrete	150 lbs/cu.ft.

Live Loads (Uniform)

Human Traffic	80 lbs/sq.ft.
Automobile Traffic	125 lbs/sq.ft.

Live Loads (Concentrated)

A 14-ton truck was thought to be the heaviest truck that would pass over the bridge. This concentration was reduced to equivalent uniform load by dividing the concentration by the plan area of the truck. Then the equivalent uniform live load =

$$\frac{28000}{10 \times 32} = 87.5 \text{ lbs/ft}^2$$

Conditions of Calculations

E_s = 30,000,000 lbs./~~ft~~in^2.

E_c = 2,000,000 lbs./~~ft~~in^2.

n = 15

f_c = 500 lbs./~~ft~~in^2. exclusive of temperature stress.

f_c = 750 lbs./~~ft~~in^2. due to a 20° F. temperature change.

f_s = 15,000 lbs./~~ft~~in^2. including a temperature change of 20° F.

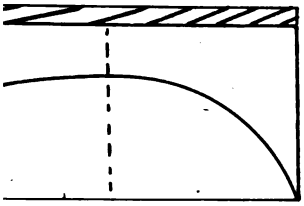
Tension in the concrete is to be neglected.

Maximum allowable shear on concrete = 75 lbs/ft^2.

Loading Systems

1. Dead Load only.
2. Dead Load plus Live Load on the whole span.
3. Dead Load plus Live Load on the center half of span to yield maximum $+M_c$.
4. Dead Load plus Live Load on the end three-eighths of the span to yield maximum $-M_c$.
5. Dead Load plus Live Load on five-eighths of the span to yield maximum $+M_s$.
6. Dead Load plus Live Load on three-eighths of the span to yield maximum $-M_s$.

CONDITIONS OF LOADING



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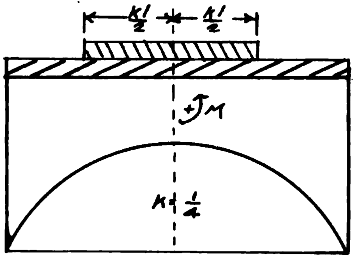

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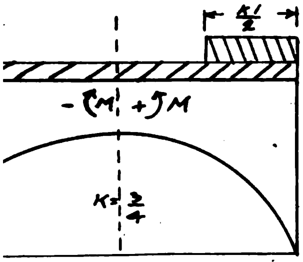
D.L.+L.L. for Max. $+M_c$



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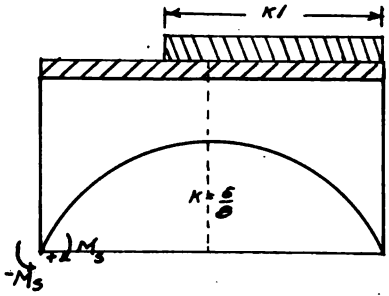
L.+L.L. for Max. $-M_c$



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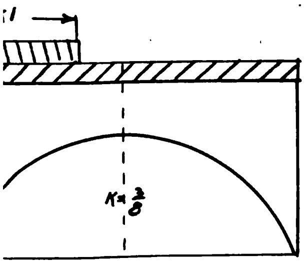

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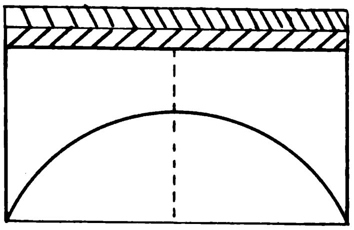

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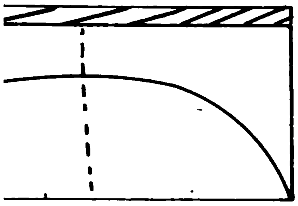
D.L.+L.L. on Whole Span

CONDITIONS OF LOADING



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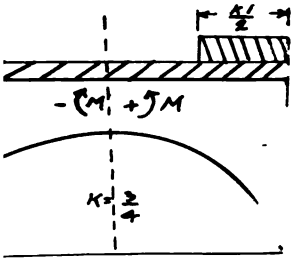

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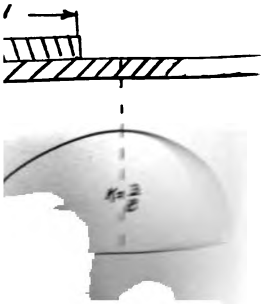

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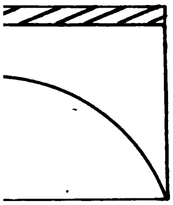
CONDITIONS OF LOADING



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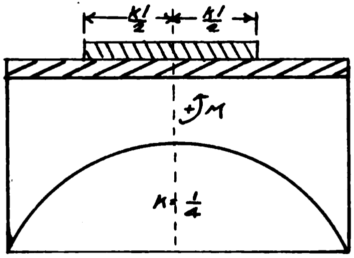

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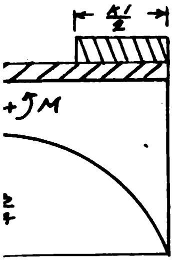

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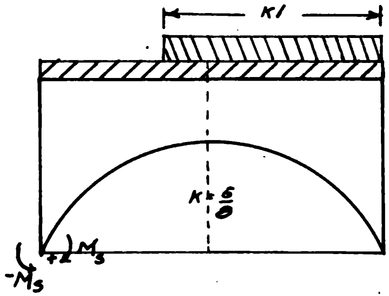
Max.- M_c



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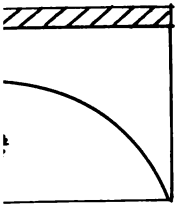
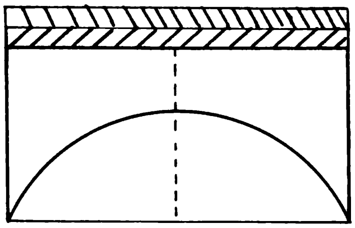


g 6
Max.-M_s



Loading 2
D.L.+L.L. on Whole Span

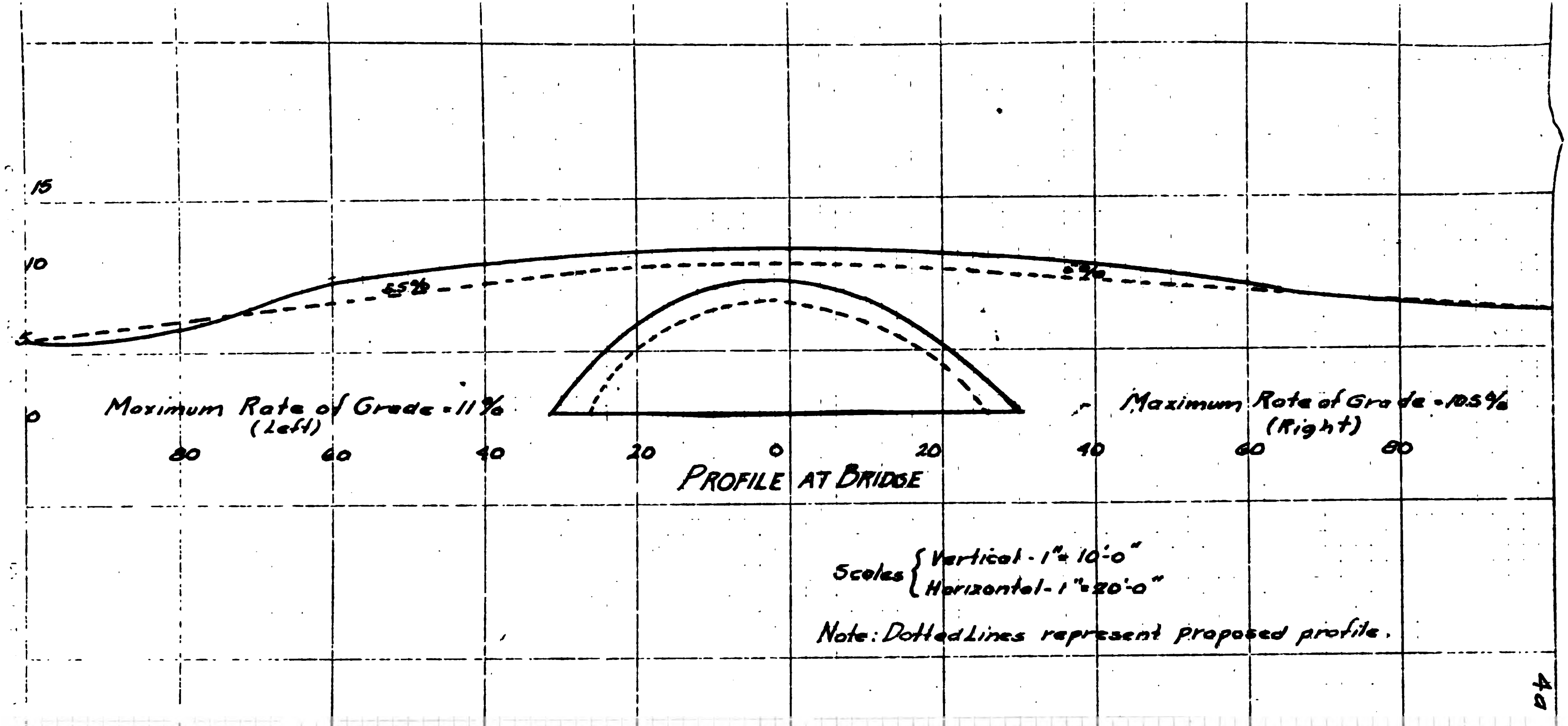
APTER III. PRELIMINARY INVESTIGATIONS

iminary investigations, consisting of a location traffic census, were made in order to determine ı, and width of the bridge, and to ascertain ns traffic might assume in the future.

tion survey furnished data from which a proreet grade and a map showing the existing bridge he survey disclosed the fact that the Yahara Lakes Mendota and Monona acted in the capacity nnel in which there was a regulated flow of wasmall rate of discharge. It was also noted that etween the two lakes carried off excessive stor-Mendota.

profile showed that the maximum grade of the exwere 11 and 10.5 per cent, respectively. Several occurred on the bridge within the last few t seemed to indicate that the bridge afforded ssing to motor vehicles. Investigation proved of a vehicle could not see traffic that was i him.

was obvious that the grade of the approaches in order to minimize the danger, a tentative e in which the rise and span of the proposed ced. But, with a decrease in rise and span, lements must be considered before such a deplace.



PROFILE AT BRIDGE

Scales { Vertical - 1" = 10'-0"
Horizontal - 1" = 20'-0"

Note: Dotted Lines represent proposed profile.

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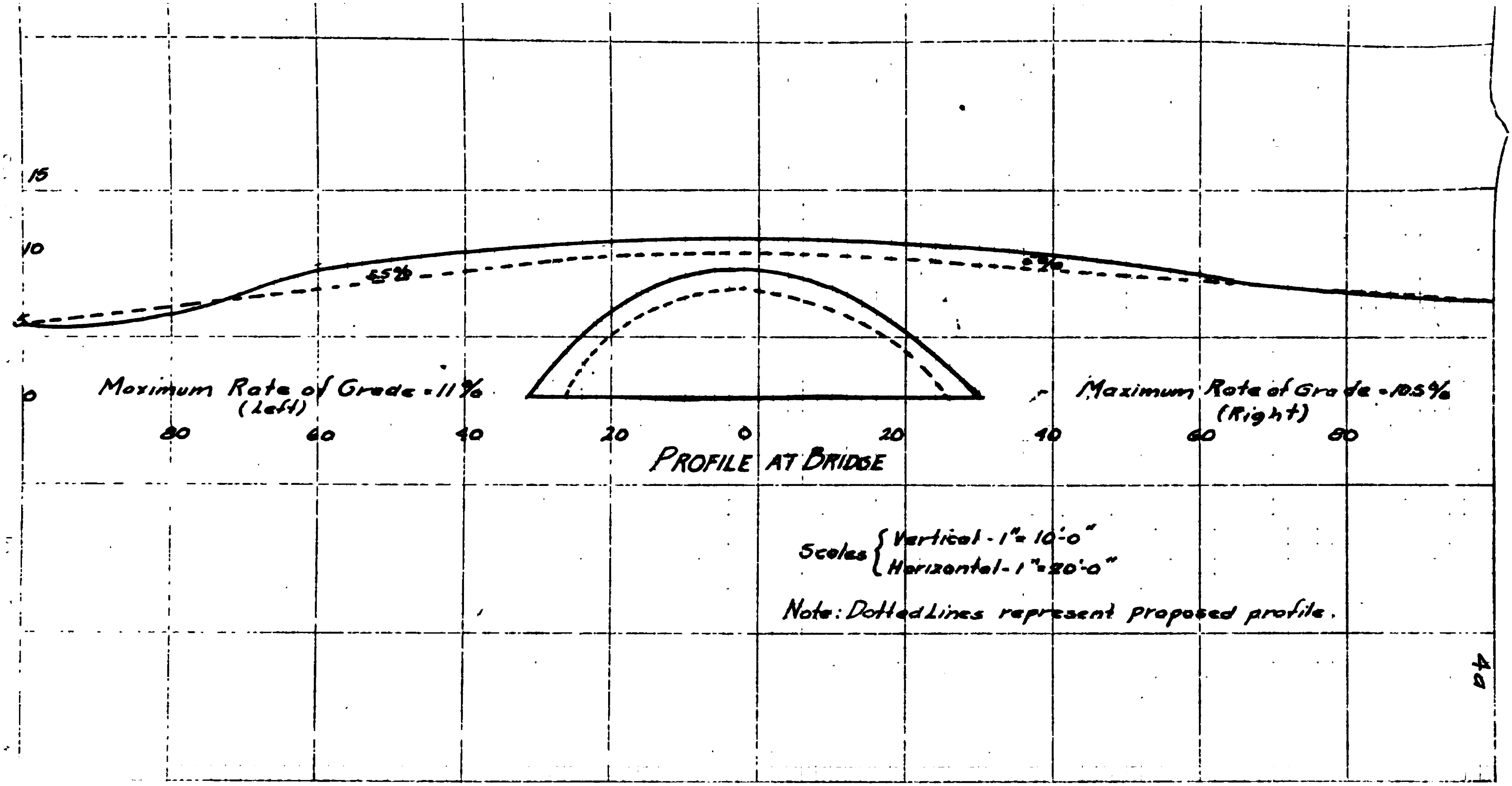


PROFILE AT BRIDGE

Scales { Vertical - 1" = 10'-0"
{ Horizontal - 1" = 20'-0"

Note: Dotted Lines represent proposed profile.

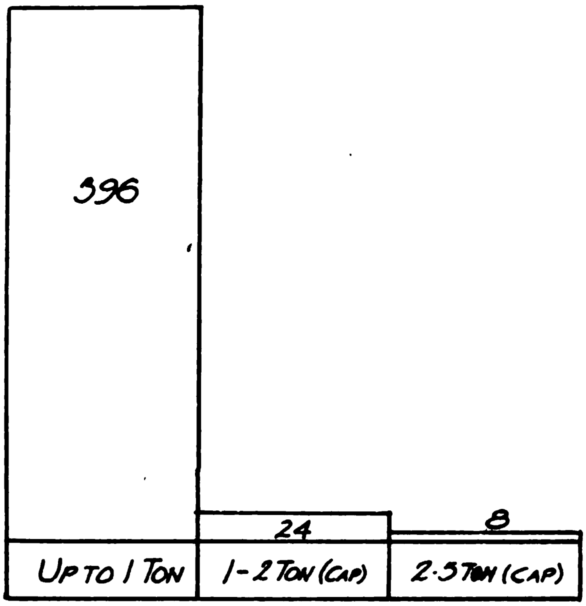
In the first place, by decreasing the span of the bridge, the abutments would tend to restrict the water in the river. This would cause an increase in the velocity of the water, and, if the river assumed an abnormal size due to flood conditions, the stability of the bridge would be threatened. Secondly, by decreasing its rise, the bridge would bar many of the lake craft which formerly used the Yahara River.

The fact that the river was an open channel with a regulated flow of water eliminated the first element from influencing the bridge design; also, since the Williamson Street bridge, two blocks away, had a smaller clearance than that of the existing bridge, the size of lake craft entering the Yahara River was limited by this bridge. Consequently, if the rise of the proposed bridge was decreased to that of the Williamson Street bridge, no limitation of the size of lake craft would be necessary, and therefore the second element would not have to be considered.

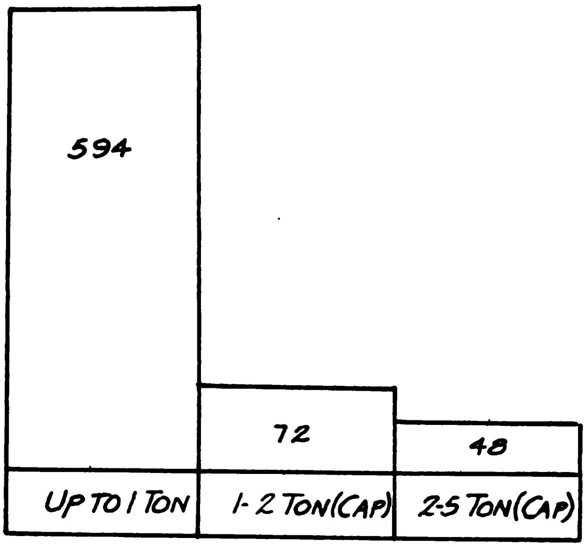
A span of fifty feet with a clearance like that of the Williamson Street bridge, and with approach grades of 5.5 and 5 per cent were chosen. It was thought that these values would reduce the danger afforded by the present crossing.

The traffic census indicated that very few concentrated loads were applied to the existing bridge, since most of the vehicles which crossed the bridge were automobiles and light trucks.

AN ESTIMATE
OF
TRAFFIC CONDITIONS ACROSS THE EXISTING RUTLEDGE STREET BRIDGE
FOR
A PERIOD OF TWELVE HOURS



NUMBER OF VEHICLES PASSING OVER BRIDGE.



TONNAGE OF VEHICLES PASSING OVER BRIDGE

The location of the bridge seemed to reveal that traffic for the most part would consist of automobiles, since the bridge lies in a residential portion of the city, and since city officials plan to make Rutledge Street a part of the Lake Monona Drive. It was thought that a 14-ton truck would provide the heaviest concentrated load to which the bridge would be subjected.

HAPTER IV. DESIGN OF THE ARCH RING

es which occur in arch bridges are caused by such as dead and live load, and such horizontal forces hange and side pressure due to earth filling. In last force may be neglected.

ination of the thickness of the arch ring and the arch axis fixes the amount of dead load, ines the position of the line of pressure with ch axis. A form of arch in which the arch axis pressure most closely approach each other is n be designed. Since live loads and temperature iable stresses, since the dead load is much live load, and since the curvature of the arch y one condition of loading, it is a general d load shall define the curvature of the arch

method is a cut and try process, the dimensions he curvature of the arch axis should be deteris of the stresses in the arch ring should be their intensities. If the allowable stresses d, the arch must be revised and the process re-

the Thickness of the Arch Ring

empirical formulae have been developed to deess of the arch at the crown, in which the live load, the fill over the crown at the exise of the arch have been made to vary with

None of these furnish an exact result, because of the many variables entering into the determination of the crown thickness. An average value derived from F. F. Weld's, D. B. Luten's, and Balet's formulae will be used to determine the thickness of the arch at the crown. A three-centered extrados and intrados will be used in the design, in which the radial thickness at the quarter point will be one and an eighth times greater than the radial thickness at the crown, and the radial thickness at the springing will be two and one-half times greater than the radial thickness at the crown.

F. F. Weld's Formula

$$d = \sqrt{S} + 0.1S + 0.005W + 0.0025P$$

where

d = the depth of the arch ring at the crown in inches

S = the clear span in feet

W = the live load in pounds per square feet

P = the weight of the dead load above the crown of the arch in pounds per square foot

Then $S = 50$ ft; $W = 125$ lbs/sq.ft; $P = \frac{6\frac{3}{4}}{12} \times 150 + 120 + 150 \times \frac{14}{12} =$ 380 lbs/sq.ft.

It was assumed the depth of the arch ring at the crown was 14".

$$d = \sqrt{50} + 0.1 \times 50 + 0.005 \times 125 + 0.0025 \times 380 = 13.65'' \text{ or } 14''$$

Since 14" was assumed, it will not be necessary to recompute P.

D.B.Luten's Formula

$$d = 4 + \frac{3S^2(r + 3F)}{4000r - S^2} + \left(\frac{L_1 S^2}{30000r} \text{ or } \frac{L_m(S+5r)}{150\ r} \right)$$

Note: Use that ~~the~~ term of the two containing L_1 and L_m that gives the greater value.

e depth of the arch ring at the crown in inches

e clear span in feet

e fill over the crown at the extrados in feet

rise from the springing line to the soffit in ft.

form live load in lbs. per square foot

moving live load that will be concentrated on

ingle path over half the span in tons of 2000#

"; $F = 1.58'$; $L_l = 125$ lbs/sq.in; $L_m = 14$ tons.

se values,

$$+ \frac{3x2500(7.5+3x1.58)}{4000x7.5 - 2500} + \left(\frac{125x2500}{30000x7.5} \text{ or } \frac{14(50+5x7.5)}{150 \times 7.5}\right)$$

$$+ 3.34 + (1.39 \text{ or } 1.09) = 8.73 \text{ or } 9 \text{ in.}$$

ches have a rise of less than one-third of the span

$$33n + \frac{0.001WS}{2r_a f_c}$$

a rise of more than one-third the span

$$33n + \frac{1}{336} \cdot \frac{W}{r_a}$$

depth of the arch ring at the crown in feet

weight in pounds of one-half of the arch, in-

iling paving and a fill less than 3.5 ft.in depth

he crown

average stress in the arch in lbs.per sq.ft.

clear span in feet

rise of the arch axis in feet

oefficient which is usually assumed for bridges

be 0.2

from Balet's formulae should be reduced 15% for

ed concrete arches.

W is given very closely by means of the following formula

$$W = (f + d + kr)\frac{S}{2} \times 130$$

where

W = the weight in pounds of one-half of the arch

f = the fill over the crown at the extrados in feet

d = the thickness of the arch ring at the crown in ft.

r = the rise from the springing line to the soffit in feet.

S = span of the arch in feet

k = a factor depending upon the ratio of the rise to the span (See the following table).

TABLE OF VALUES FOR K

Ratio of Rise to Span	.13	.14	.155	.17	.185	.20	.215	.23	.243	.257
K	.34	.33	.32	.31	.30	.29	.28	.27	.26	.25

In the formula for the value of W

$f = 1.58'$; $d = 12''$ (assumed); $r = 7.5'$; $S = 50'$; $k = .323$

$W = (1.58 + 1 + .323 \times 7.5) 25 \times 130 = 16,850$ lbs.

Then in Balet's formula, where arches have a rise of less than one-third the span

$n = 0.2$; $W = 16,850$#s; $S = 50'$; $r_a = r - \frac{1}{2}d - 7.5 - .5 = 7'$

$f_c = 250$#s/sq.in. = one-half the maximum allowable stress

$$d = .85 \left(3.333 \times 0.2 + \frac{0.001 \times 16850 \times 50}{7 \times 250}\right) = .974' = 11.7 \text{ in.}$$

Taking an average if the three values determined for d,

$$d = \frac{13.65 + 8.73 + 11.7}{3} = 11.36''$$

In the design, 12 in. will be used for the thickness of the arch ring at the crown, and the vertical depth at the springing line will be made 36 in.

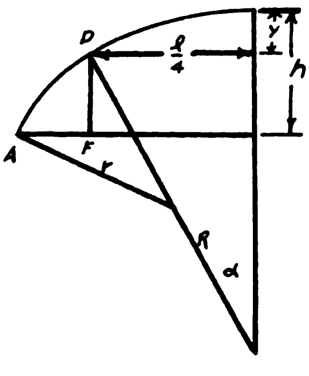
Determination of the Curvature of the Arch Axis

A trial arch was assumed, in which the intrados, the arch axis, and the extrados were three centered curves. The horizontal distance of the arch from the springing to the crown, was divided into twenty equal divisions, ten divisions on each side of the crown, and the amounts of the dead loads and their action lines were determined.

Since the dead load was much greater than the live load, a deflection curve for the live load would not deviate much from the dead load deflection curve. This approximates the condition for moments in a three hinge arch. Then summation of moments at the crown and at the springing were assumed equal to zero. By taking a summation of moments at the springing, the true pole for the equilibrium polygon was determined.

A force diagram for half the arch was drawn, from which the equilibrium was made to pass through the arch axis at the crown. The equilibrium polygon so closely approximated the form of the arch axis, that the curvature of the arch axis was deemed satisfactory.

Determination of the Radii of a Three-Centered Curve for the Intrados, Arch Axis, and the Extrados.



$$R = \frac{y^2 + l^2/16}{2y} \tag{1}$$

$$r = \frac{AF^2 + FD^2}{FD\cos\alpha - AF\sin\alpha} \times \frac{1}{2} \tag{2}$$

In order to solve for the radii, it was necessary to determine the value of y.

A general equation

$$y = \frac{k^2 h\,[1 + k^n(2K/(n+1)(n+2))]}{[1 + (2K/(n+1)(n+2))]}$$

was developed by Professor W. S. Kinne at the University of Wisconsin.

h = the rise from the springing line to the soffit in feet

k = the ratio of the distance from the point in question to the crown to half the span.

K = a constant determined by the relationship $K = \frac{w_s}{w_c} - 1$.

w_s = the dead load at the springing.

w_c = the dead load at the crown.

n = an average value of 3.

If $n = 3$; $k = \frac{1}{2}$, then the general equation can be expressed as

$$y = \frac{h}{4}\frac{(1 + 0.0125K)}{(1 + 0.1K)} \qquad 3)$$

If w_c is the dead load above the crown,

$$w_c = 150x1 + 120x1 + 150x\frac{6\frac{3}{4}}{12} = 355\#s.$$

w_s, the dead load at the springing, is dependent upon the weight of the arch at the springing, the fill, and the pavement above the arch at the springing.

The vertical depth of the concrete at the springing is 3'2"; that of the fill, 8' 0"; and that of the pavement, 6¾". Then,

$$w_s = 3.17 \text{ x } 150 + 8x120 + 150x6\tfrac{3}{4} = 1520\#s.$$

$$K = \frac{w_s}{w_c} - 1 = \frac{1520}{355} - 1 = 3.29$$

$$h = 7.5 \text{ feet}$$

Substituting these values in equation 3), $y = 1.47$ ft. (arch axis value for y)

TABLE
FOR THE COMPUTATION OF
CED LOAD CONTOURS AND DEAD LOAD

Depth of Earth Fill.	Depth of Arch Ring.	Reduced Depth of Earth Fill.	Reduced Depth of Pavement.	Equivalent Depth of Concrete.	Ave. Equiv. Depth of Concrete.	Value of Dead Load.
1.00	1.00	0.80	0.56	2.38		
1.05	1.00	0.84	0.56	2.40	2.38	893
1.25	1.02	1.00	0.56	2.58	2.49	934
1.58	1.05	1.26	0.56	2.87	2.72	1021
1.98	1.09	1.58	0.56	3.23	3.05	1142
2.55	1.12	2.04	0.56	3.72	3.48	1301
.21	1.28	2.57	0.56	4.41	4.06	1524
.09	1.49	3.27	0.56	5.32	4.86	1825
10	1.88	4.08	0.56	6.52	5.92	2219
29	2.39	5.03	0.56	7.98	6.25	2342
70	3.20	6.15	0.56	9.91	8.94	3350

For the intrados, $x = 12.3'$; $y = 1.60'$; and $h = 8.9'$.

For the extrados, $x = 12.6'$; $y = 1.51'$; and $h = 6.62'$.

The radii for the arch axis were determined by substituting in Equations 1) and 2).

$$R = \frac{1.47^2 + 156}{2 \times 1.47} = 53.75 \text{ ft.}$$

$$\sin \alpha = \frac{12.5}{53.75} = .2325$$

$$\cos \alpha = \frac{53.75-1.47}{53.75} = .972$$

$$r = \frac{\frac{1}{2}(156+36.5)}{6.03 \times .972 - 12.5 \times .2325} = 33.1 \text{ ft.}$$

By a similar substitution, the radii for the intrados are $R = 50.0$ ft. and $r = 27.2$ ft. Also the radii for the extrados are $R = 54.5$ ft. and $r = 43.4$ ft.

Reduced Load Contours, Amount and Action Lines of the Dead Loads

The horizontal distance between the left springing and the crown was divided into ten equal divisions by means of vertical lines extending from the intrados to the top of the ballast. The ordinates of the "Reduced Load Contour", which were to be measured above the intrados, were determined, and the amount of the dead load for each division was evaluated by means of Table B.

The "Reduced Load Contour" was plotted, and the centers of the ten equal divisions were ascertained. In order to make a force diagram, it was necessary to determine the true pole distance.

Determination of the True Pole for the Equilibrium Polygon.

H is the true pole distance. Taking a summation of moments about the springing, $M_A = 0$, H was analytically determined.

3350 x 1.25 = 4180
2342 x 3.75 = 8780
2219 x 6.25 = 13840
1825 x 8.75 = 15950
1524 x 11.25 = 17180
1301 x 13.75 = 17890
1142 x 16.25 = 18590
1021 x 18.75 = 19150
933.5x 21.25 = 19800
893 x 23.75 = 21190
7.5H = 156550
H = 20870

To check this value of H, the center of gravity of all the vertical forces was determined. Since there was a summation of moments at A and at C equal to zero, a horizontal line from C was drawn to cut the resultant vertical force. If the line connecting A to this point was parallel to the line connecting the vertical forces and H in the equilibrium polygon, H was the true pole distance.

Taking $M_A = 0$ for the vertical forces,

16550 . 5xX = 156550
X = 9.46 ft.

Since the lines mentioned above were parallel, H was the true pole distance. The strings of the equilibrium polygon were then drawn in. It was found that the strings of the equilibrium polygon passed through the arch axis at the springing and at the crown, and that the equilibrium polygon approached the arch axis very closely.

Analysis of the Stresses in the Arch Ring

At this point, a determination was made to see whether or not the allowable values were exceeded when the arch was loaded to yield maximum stresses. For the general theory of arch bridges, see "Modern Framed Structures", Part II, Chapter IV, and for the application of the theory see "Principles of Reinforced Concrete Construction", Chapter VIII.

termined by the preliminary work was laid out to 2'0". The arch ring was divided into eight di-

$\frac{s}{I}$ was a constant.

ep in dividing the arch ring into eight divisions e length of the arch axis between the crown and ng the center of the extrados curve with the in- vertical line through the springing and the ex-

the Length of the Arch Axis

$$\theta_1 = \sin^{-1}\frac{BD}{BO} = \frac{12.5}{53.75} = 13^\circ - 25'.$$

$$\theta_2 = 2\sin^{-1}\frac{\text{Chord BC}}{2\ BP} = 2\sin^{-1}\frac{13.2}{2\times 33.1} = 23^\circ - 0'$$

$$\text{The arc AB} = \frac{13.42}{360} \times 2\pi \times 53.75 = 12.6 \text{ ft.}$$

$$\text{The arc BC} = \frac{23}{360} \times 2\pi \times 33.1 = 13.3 \text{ ft.}$$

$$\text{Length of arc AC} = 25.90 \text{ ft.}$$

f the arch axis, AC, was divided up into ten after which the radial thickness at the crown, g, and at the intermediate points, were scaled. e listed in Table No. 1.

values in Table No. 1, a Curve 1 was plotted, iation in radial depth of the arch ring between he springing line. Values from this curve were ure computations, in which the radial depth of as involved.

TABLE #1

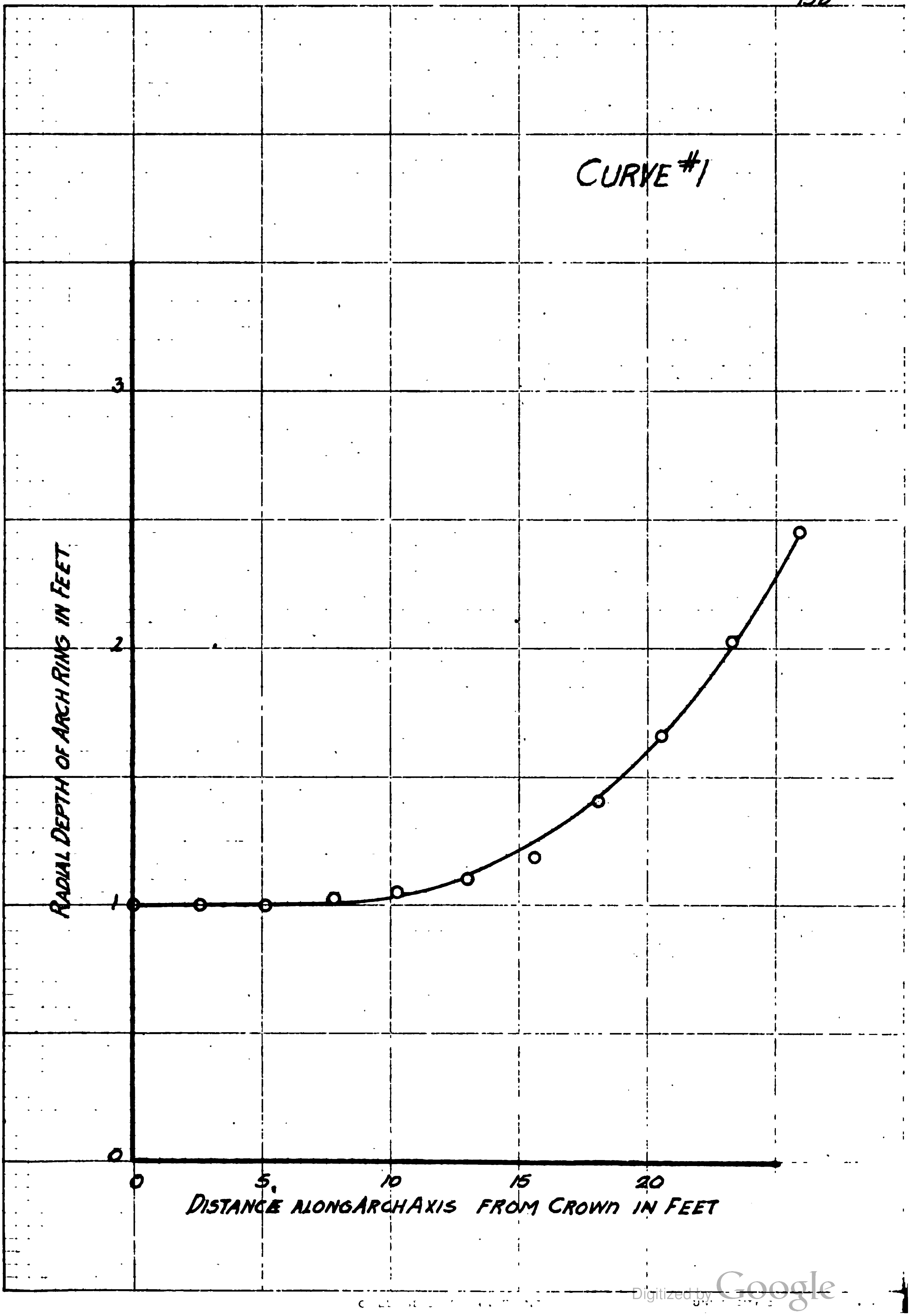
Number of Point	Distance from Crown	Radial Depth Scaled	Corrected from Curve I.
Crown	0.00	1.00	1.00
1	2.59	1.00	1.00
2	5.18	1.00	1.00
3	7.77	1.02	1.00
4	10.36	1.04	1.03
5	12.95	1.10	1.10
6	15.54	1.18	1.22
7	18.13	1.40	1.40
8	20.72	1.68	1.64
9	23.31	2.04	1.98
Springing	25.90	2.45	2.45

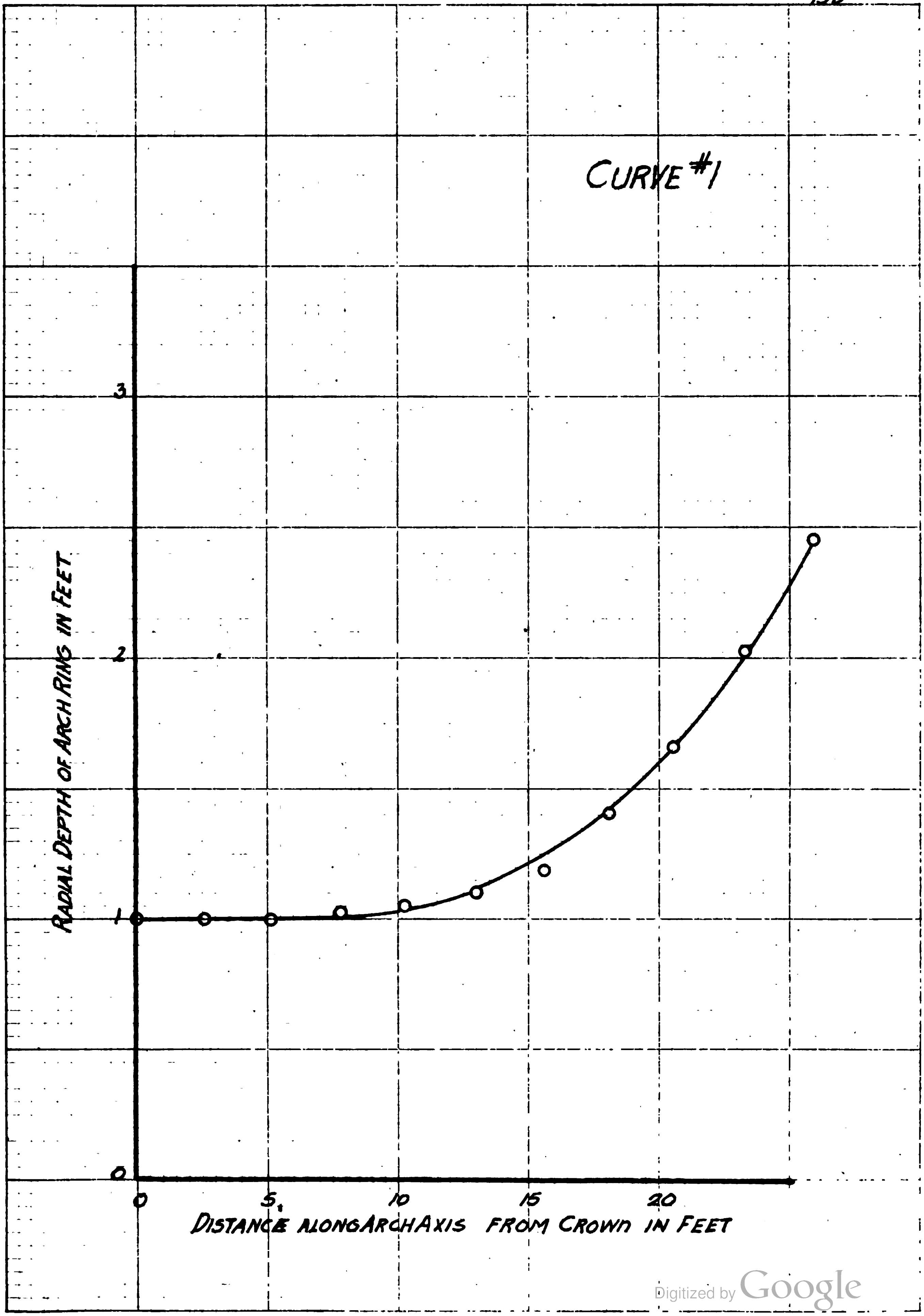
TABLE #2
COMPUTATION FOR VALUE OF i_a

(1) Point	Radial Depth of Center h	(3) h^3	(4) $I_c = h^3/12$	(5) $h/2$	(6) $h/2 - d'$	(7) $(h/2 - d')^2$	(8) $15 I_s = 2nA(7)$	(9) $I = I_c + 15 I_s$	(10) $i = 1/I$
Crown	1.00	1.00	0.083	.50	.25	.063	.013	.096	10.41
1	1.00	1.00	0.083	.50	.25	.063	.013	.096	10.41
2	1.00	1.00	0.083	.50	.25	.063	.013	.096	10.41
3	1.00	1.08	0.083	.50	.25	.063	.013	.096	10.41
4	1.03	1.09	0.091	.52	.27	.073	.015	.106	9.43
5	1.10	1.33	0.111	.55	.30	.090	.019	.130	7.69
6	1.22	1.82	0.152	.61	.36	.130	.027	.179	5.59
7	1.40	2.74	0.228	.70	.45	.203	.043	.271	3.69
8	1.64	4.41	0.368	.82	.57	.325	.068	.436	2.29
9	1.98	7.76	0.646	.99	.74	.548	.115	.761	1.31
Springing.	2.45	14.71	1.225	1.22	.97	.941	.198	1.423	0.70

$d' = 3'' = .25'$; $A = .007 \times 1 \times 1 = .007\ \text{ft}^2$; $2nA = 2 \times 15 \times .007 = .21\ \text{ft}^2$

$i_a = \frac{72.34}{11} = 6.576$


CURVE #1
RADIAL DEPTH OF ARCH RING IN FEET
0
1
2
3
0
5
10
15
20
DISTANCE ALONG ARCH AXIS FROM CROWN IN FEET


CURVE #1
RADIAL DEPTH OF ARCH RING IN FEET
DISTANCE ALONG ARCH AXIS FROM CROWN IN FEET
0
1
2
3
0
5
10
15
20

The value of i_a was computed (See Art. 178, Turneaure and Maurer), by means of Table No. 2. The moment of inertia at any section is equal to $I = I_c - nI_s$, where I_s and I_c are the moment of inertia of the steel and the concrete respectively. The amount of steel reinforcement along the intrados and the extrados was assumed to be 0.7 per cent of the area of the cross section of the arch rib at the crown.

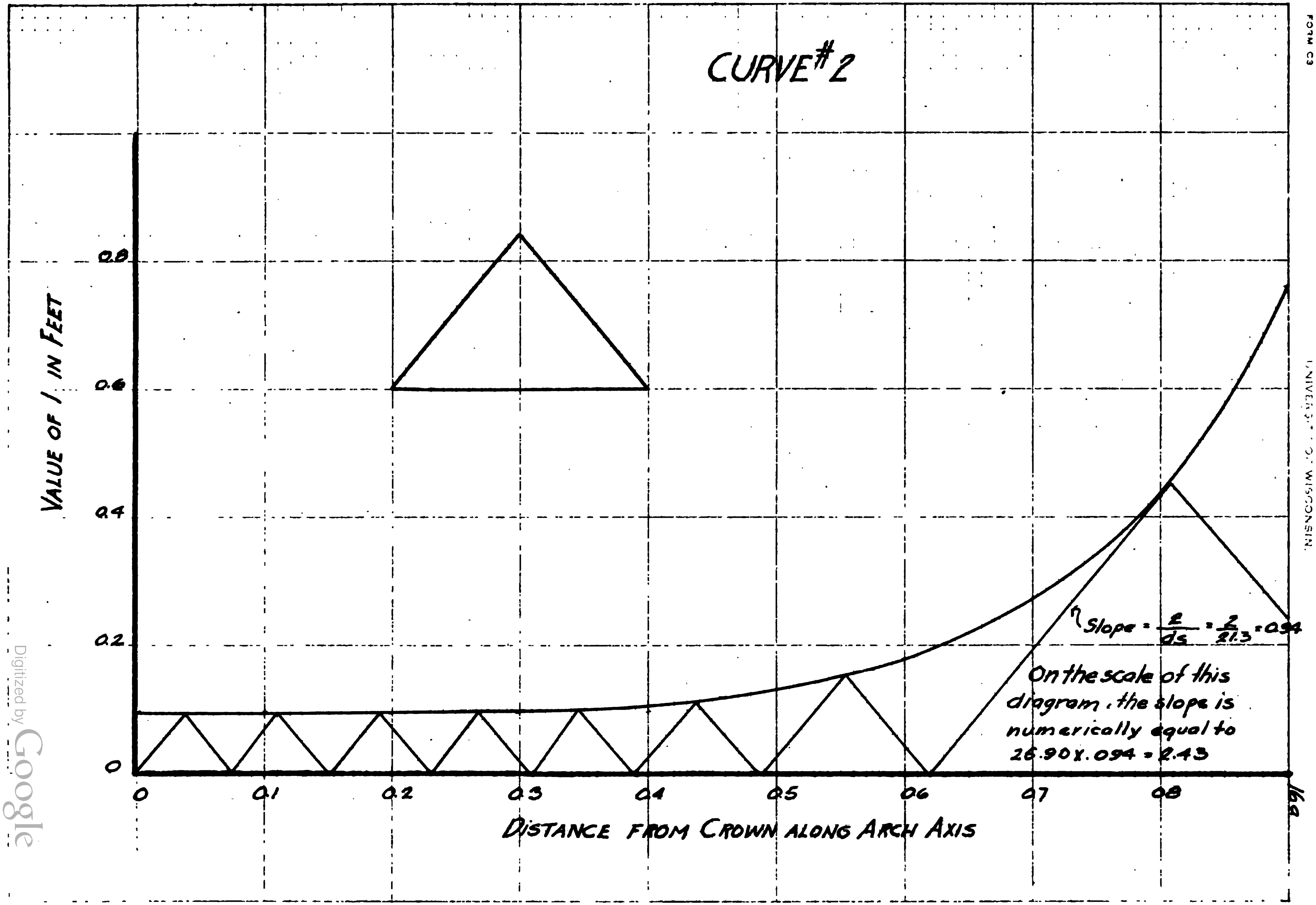
The constant $\frac{ds}{I}$ was determined from the formula $\frac{ds}{I} = \frac{si_a}{N}$ (See page 342, Turneaure and Maurer).

$$\frac{ds}{I} = \frac{25.9 \times 6.576}{8} = 21.3$$

N = the desired number of divisions in half the span.

The lengths of the eight $\frac{ds}{I}$ constant divisions was determined by the following graphical method: A curve was plotted, showing the variation of the moment of inertia between the crown and the springing. The moments of inertia were plotted as the ordinates and the distances as the abscissae. These lengths are listed in Table No. 3.

The lengths of the eight divisions were laid off along the arch axis, and the center of each division was located. After these points had been numbered one to eight, starting from the crown, the values of x and y, the co-ordinates of these center points and of the springing point on the arch axis were scaled, the crown point of the arch axis being used as the origin. These values were also tabulated in Table No. 3.



Slope $= \frac{2}{ds} = \frac{2}{21.3} = .094$

On the scale of this diagram, the slope is numerically equal to $26.90 \times .094 = 2.43$

TABLE #3
LENGTHS OF DIVISIONS TO MAKE $\frac{ds}{I}$ CONSTANT AND COORDINATES OF DIVISION CENTERS

Number of Divisions	Length	Coordinates	
		X	Y
Crown		0.00	0.00
1	1.98	0.99	0.04
2	1.98	2.98	0.10
3	1.98	4.95	0.24
4	1.98	6.92	0.45
5	2.12	9.00	0.79
6	2.59	11.30	1.25
7	3.37	14.09	1.95
8	9.90	20.20	4.42
Springing		24.30	7.16

TABLE #4
VALUE OF DEAD LOADS AND COORDINATES OF THEIR ACTION LINES.

Load Number.	X-Coordinate.	Amount of Load.
P_1	1.25	893
P_2	3.75	934
P_3	6.25	1021
P_4	8.75	1142
P_5	11.25	1301
P_6	13.75	1524
P_7	16.25	1825
P_8	18.75	2219
P_9	21.25	2342
P_{10}	23.75	3350

e values of the dead loads and the x-coordinates of their
lines were obtained from the preliminary arch, and were
in Table No. 4.

he next important step in the analysis of the stresses
arch ring was to assume that the arch was cut at the
forming two cantilevers extending from the abutments.
ents "m" were computed from the dead load. Values of m_l
nted the moments in the left half at each of the center
from one to eight, inclusive, due to all the loads lying
the point in question and the crown, while the values
epresented the moments in the right half. Since the arch
metrical, $m_l = m_r$ for the dead load.

Since the moments at the center of the $\frac{ds}{I}$ constant di-
s were desired, it was necessary to compute the moments
nteen points, since loads were considered as being ap-
at the intermediate points. It was then possible to se-
ne desired moments. Table No. 5 was the moment table
e dead load only. A check on the table consisted in the
at the springing being equal to the sum of the loads
lied by the distance from the center of gravity from the
ing. Cantilever moments for the six conditions were com-
and moment tables were constructed.

TABLE # 5.

CANTILEVER MOMENT TABLE FOR DEAD LOAD.

Distance from rown.	Load at Point	Sum of the Loads	Distance between Points	Increment of Moment	Moment	Point
0.00	000	000	0.00	000	.000	
0.99	000	000	0.99	000	000	1
1.25	893	893	0.26	232	232	
2.98	000	893	1.73	1542	1774	2
3.75	934	1827	0.77	1404	3178	
4.95	000	1827	1.20	2189	5367	3
6.25	1021	2848	1.30	3700	9067	
6.92	0000	2848	0.67	1909	10976	4
8.75	1142	3990	1.83	7296	18272	
9.00	0000	3990	0.25	996	19268	5
11.25	1301	5291	2.25	11880	31148	
11.30	0000	5291	0.05	264	31412	6
13.75	1524	6815	2.45	16700	48112	
14.09	0000	6815	0.34	2318	50430	7
16.25	1825	8640	2.16	18640	69070	
18.75	2219	10859	2.50	27100	96170	
20.20	0000	10859	1.45	15730	111900	8
21.25	2342	13201	1.05	13850	125750	
23.75	3350	16551	2.50	41400	167150	
24.30	16551	16551	.55 24.30	9100 176250	176250	

he sum of the loads in column 3 equals the sum opposite olumn 4.

he sum of the distance between points, column 5, equals distance from crown to springing, column 2.

e sum of the increments of moment, column 6, equals the ent at the springing, column 7.

TABLE #6

OMENT TABLE FOR LIVE LOAD ON WHOLE SPAN

- 50'-0" LOAD PER FOOT - 125 LBS.

LOAD = 125X50 = 312.5 LBS. LEFT HALF SPAN.

Sum of oads	Distance between Points	Increment of Moment	Moment (ft#s)		Point.
			LL Only	L.L. + D.L.	
000	0.00	000	000	000	
000	0.99	000	000	000	1
313	0.26	081	081	313	
313	1.73	541	622	2396	2
625	0.77	481	1103	4281	
625	1.20	749	1852	7219	3
938	1.30	1217	3069	12136	
938	0.67	628	3697	14673	4
1250	1.83	2285	5982	24254	
1250	0.25	312	6294	25562	5
1563	2.25	3518	9812	40960	
1563	0.05	078	9890	41302	6
1875	2.45	4590	14480	62592	
1875	0.34	638	15118	65548	7
2188	2.16	4724	19842	88912	
2500	2.50	6250	26092	122262	
2500	1.45	3625	29717	141617	8
2813	1.05	2955	32672	158422	
3125	2.50	7810	40482	207632	
3125	0.55 24.30	1718 42200	42200	218450	

TABLE #6

ILEVER MOMENT TABLE FOR LIVE LOAD ON WHOLE SPAN

D LENGTH - 50'-0" LOAD PER FOOT - 125 LBS.

T OF EACH LOAD = 125X50 = 312.5 LBS. LEFT HALF SPAN.

st. om own	Load at Point	Sum of Loads	Distance between Points	Increment of Moment	Moment (ft#s)		Point.
					LL Only	L.L. + D.L.	
.00	0.0	000	0.00	000	000	000	
.99	0.0	000	0.99	000	000	000	1
.25	312.5	313	0.26	081	081	313	
.98	000.0	313	1.73	541	622	2396	2
.75	312.5	625	0.77	481	1103	4281	
.95	000.0	625	1.20	749	1852	7219	3
5.25	312.5	938	1.30	1217	3069	12136	
6.92	000.0	938	0.67	628	3697	14673	4
8.75	312.5	1250	1.83	2285	5982	24254	
9.00	000.0	1250	0.25	312	6294	25562	5
1.25	312.5	1563	2.25	3518	9812	40960	
1.30	000.0	1563	0.05	078	9890	41302	6
3.75	312.5	1875	2.45	4590	14480	62592	
4.09	000.0	1875	0.34	638	15118	65548	7
6.25	312.5	2188	2.16	4724	19842	88912	
8.75	312.5	2500	2.50	6250	26092	122262	
20.20	000.0	2500	1.45	3625	29717	141617	8
1.75	312.5	2813	1.05	2955	32672	158422	
23.25	312.5	3125	2.50	7810	40482	207632	
4.30	3125.0	3125	0.55 24.30	1718 42200	42200	218450	

TABLE #7

CANTILEVER MOMENT TABLE FOR MAXIMUM $+M_c$

DED LENGTH - 12.5 FT. LOAD/FT. = 125 LBS. L.H SPAN

istance from rown.	Load at Point.	Sum of Loads	Dist. b'tw'n Points	Increment of Moment	Moment		Point.
					L.L. Only	D.L. + L.L.	
0.00	0	0	0.00	0	0	0	
0.99	0	0	0.99	0	0	0	1
1.25	312.5	312.5	0.26	81	81	313	
2.98	0	312.5	1.73	541	622	2396	2
3.75	312.5	625.0	0.77	481	1103	4281	
4.95	0	625.0	1.20	749	1852	7219	3
6.25	156.3	781.3	1.30	1015	2867	11934	
6.92	0	781.3	0.67	523	3390	14366	4
8.75	0	781.3	1.83	1430	4820	23092	
9.00	0	781.3	0.25	195	5015	24283	5
11.25	0	781.3	2.25	1758	6773	37921	
11.30	0	781.3	0.05	39	6812	38224	6
13.75	0	781.3	2.45	1912	8724	56936	
14.09	0	781.3	0.34	266	8990	59420	7
16.25	0	781.3	2.16	1688	10678	79748	
18.75	0	781.3	2.50	1952	12630	108800	
20.20	0	781.3	1.45	1132	13762	125662	8
21.25	0	781.3	1.05	820	14582	140332	
23.75	0	781.3	2.50	1952	16534	183684	
24.30	0 781.3	781.3	0.55	16577 43	16577	192827	

TABLE #8

CANTILEVER MOMENT TABLE FOR MINIMUM $-M_C$

ADED LENGTH - 37.5 FT. LOAD/FT. = 125 LBS. L.H.SPAN

Distance from Crown	Load at Point	Sum of Loads	Distance between Points	Increment of Moment	Moment		Point
					D.L. Only	D.L. + L.L.	
0.00	0	0	0.00	0	0	0	
0.99	0	0	0.99	0	0	0	1
1.25	0	0	0.26	0	0	232	
2.98	0	0	1.73	0	0	1774	2
3.75	0	0	0.77	0	0	3178	
4.95	0	0	1.20	0	0	5367	3
6.25	156.3	156.3	1.30	213	213	9280	
6.92	0	156.3	0.67	105	318	11294	4
8.75	312.5	4688	1.83	856	1174	19446	
9.00	0	468.8	0.25	117	1291	20559	5
11.25	312.5	781.3	2.25	1753	3050	34198	
11.30	0	781.3	0.05	39	3089	34501	6
13.75	312.5	10938	2.45	2680	5769	53881	
14.09	0	10938	0.34	372	6141	56571	7
16.25	312.5	1406.3	2.16	3030	9171	78241	
18.75	312.5	1718.8	2.50	4290	13461	109631	
20.20	0	1718.8	1.45	2470	15931	127831	8
21.25	312.5	2031.3	1.05	2130	18061	143811	
23.75	312.5	2343.8	2.50	5870	23931	191081	
24.30	0 ~~2343.8~~	2343.8	0.55	25221. 1290	25221	201471	

There was no need to compute the cantilever moments for maximum $+M_s$ and maximum $-M_s$, since these moments were combinations of moments already computed. Moments for maximum $+M_s$ (left half span) were the same as for maximum $+M_c$ (left half span), while the right half span moments were equal to those of the dead load plus the live load (either half span). Moments for maximum $-M_s$ (left half span) coincided with those for maximum $-M_c$ (left half span), while the right half span moments were equal to the dead load only (either half span).

Summation tables were made for each of the six systems of loadings, and the thrust, shear, and moment at the crown was computed for each case. The values m_l and m_r were obtained directly from either Tables 5, 6, 7, or 8, depending upon the loading under consideration. The values x and y in all six tables are the x- and y- coordinates of the centers of the $\frac{ds}{I}$ constant divisions.

H_0, V_0, and M_0 for each condition of loading were determined with the values obtained from Tables 9, 10, 11, 12, 13, and 14.

Case I - Dead Load only.
Case II - Dead Load plus Live Load on the Whole Span.
Case III - Dead Load plus Live Load for Maximum $+M_c$.
Case IV - Dead Load plus Live Load for Maximum $-M_c$.
Case V - Dead Load plus Live Load for Maximum $+M_s$.
Case VI - Dead Load plus Live Load for Maximum $-M_s$.

TABLE #9

SUMMATION TABLE DEAD LOAD ONLY

Point	X	Y	X^2	Y^2	M_R (-)	M_L(-)	$M_R + M_L$ (-)	$(M_R+M_L)Y$	$M_R - M_L$	$(M_R-M_L)X$
1	0.99	0.04	0.98	.00	0		0	0		
2	2.98	0.10	8.88	.01	1774	SAME AS M_R	3548	355		
3	4.95	0.24	24.50	.06	5367		10734	2560		
4	6.92	0.45	47.89	.20	10976		21952	9870		
5	9.00	0.79	81.00	.62	19268		38536	30400		
6	11.30	1.25	127.69	1.56	31412		62824	78400		
7	14.09	1.95	198.00	3.80	50430		100860	196400		
8	20.20	4.42	408.04	19.54	111900		223800	988000		
Σ	70.53	9.24	896.98	25.79	-231127		-462254	-1305985		

$$H_o = \frac{N\Sigma my - \Sigma m \Sigma y}{2[(\Sigma y)^2 - N\Sigma y^2]} = \frac{8(-1305985)-(-462254)9.24}{2\,[(9.24)^2 - 8 \times 25.79]} = +25450 \text{ LBS}$$

$$V_o = \Sigma(M_R - M_L)x / 2\Sigma x^2 = 0 \qquad M_o = -\frac{\Sigma m + 2H_o\Sigma y}{2N} = \frac{-462254 + 2 \times 25450 \times 9.24}{16} = -453^{\#s}$$

TABLE #10

SUMMATION TABLE LIVE LOAD ON WHOLE SPAN.

Point	X	Y	X^2	Y^2	M_R(-)	M_L(-)	$M_R + M_L$ (-)	$(M_R+M_L)Y$ (-)	$M_R - M_L$	$(M_R-M_L)X$
1	0.99	0.04			0		0	0		
2	2.98	0.10	SAME AS TABLE #9	SAME AS TABLE #9	622		1244	124		
3	4.95	0.24			1852	SAME AS M_R	3704	889		
4	6.92	0.45			3390		6780	3050		
5	9.00	0.79			5015		10030	7930		
6	11.30	1.25			6812		13624	17040		
7	14.09	1.95			8990		17980	35100		
8	20.20	4.42			13762		27524	121500		
Σ	70.53	9.24	896.98	25.79	- 40443		-80886	-185633		

$$H_o = \frac{[8(-185633)-(-80886)9.24}{-242.4} = 3050 \text{ LBS.} \qquad V_o = 0$$

$$M_o = \frac{-[-80886 + (2 \times 3050 \times 9.24)]}{2 \times 8} = 962 \text{ FT.LBS.}$$

TABLE #11

SUMMATION TABLE — LIVE LOAD FOR MAX. $+M_c$

Point	X	Y	X^2	Y^2	$M_R(-)$	$M_L(-)$	M_R+M_L (-)	$(M_R+M_L)Y$ (-)	M_R-M_L	$(M_R-M_L)X$
1	0.99	0.04			0		0	0		
2	2.98	0.10			622		1244	124		
3	4.95	0.24	SAME AS TABLE #9	SAME AS TABLE #9	1852		3704	889		
4	6.92	0.45			3390	SAME AS M_R	6780	3050		
5	9.00	0.79			5015		10030	7930		
6	11.30	1.25			6812		12624	17040		
7	14.09	1.95			8990		17980	35100		
8	20.20	4.42			13762		27524	121500		
Σ	70.53	9.24	896.98	25.79	-40443		-80886	-185633		

$$H_o = \frac{[8(-185633)-(-80886 \times 924)}{-242.4} = 3050 \text{ LBS.} \qquad V_o = 0$$

$$M_o = -[-80886 + (2 \times 3050 \times 9.24)]/16 = 962 \text{ FT.LBS.}$$

TABLE #12

SUMMATION TABLE — LIVE LOAD FOR MAX. $-M_c$

Point	X	Y	X^2	Y^2	$M_R(-)$	$M_L(-)$	M_R+M_L (-)	$(M_R+M_L)Y$ (-)	M_R-M_L	$(M_R-M_L)X$
1	0.99	0.04			0		0	0		
2	2.98	0.10	SAME AS TABLE #9	SAME AS TABLE #9	0		0	0		
3	4.95	0.24			0	SAME AS M_R	0	0		
4	6.92	0.45			318		636	286		
5	9.00	0.79			1291		2582	2040		
6	11.30	1.25			3089		6178	7720		
7	14.09	1.95			6141		12282	23900		
8	20.20	4.42			15931		31862	140500		
Σ	70.53	9.24	896.98	25.79	-26770		-53540	-174446		

$$H_o = \frac{[8(-174446)-(-53540)9.24]}{-242.4} = 3710 \text{ LBS.} \qquad V_o = 0$$

$$M_o = -[-53540 + (2 \times 3710 \times 9.24)]/16 = -935 \text{ FT.LBS.}$$

TABLE #11

SUMMATION TABLE — LIVE LOAD FOR MAX. $+M_c$

Point	X	Y	X^2	Y^2	$M_R(-)$	$M_L(-)$	M_R+M_L (−)	$(M_R+M_L)Y$ (−)	M_R-M_L	$(M_R-M_L)X$
1	0.99	0.04			0		0	0		
2	2.98	0.10			622		1244	124		
3	4.95	0.24	SAME AS TABLE #9	SAME AS TABLE #9	1852		3704	889		
4	6.92	0.45			3390	SAME AS M_R	6780	3050		
5	9.00	0.79			5015		10030	7930		
6	11.30	1.25			6812		12624	17040		
7	14.09	1.95			8990		17980	35100		
8	20.20	4.42			13762		27524	121500		
Σ	70.53	9.24	896.98	25.79	-40443		-80886	-185633		

$$H_o = \frac{[8(-185633)-(-80886\times 924)}{-242.4} = 3050 \text{ LBS.} \qquad V_o = 0$$

$$M_o = -[-80886+(2\times 3050\times 9.24)]/16 = 962 \text{ FT. LBS.}$$

TABLE #12

SUMMATION TABLE — LIVE LOAD FOR MAX. $-M_c$

Point	X	Y	X^2	Y^2	$M_R(-)$	$M_L(-)$	M_R+M_L (−)	$(M_R+M_L)Y$ (−)	M_R-M_L	$(M_R-M_L)X$
1	0.99	0.04			0		0	0		
2	2.98	0.10	SAME AS TABLE #9	SAME AS TABLE #9	0		0	0		
3	4.95	0.24			0	SAME AS M_R	0	0		
4	6.92	0.45			318		636	286		
5	9.00	0.79			1291		2582	2040		
6	11.30	1.25			3089		6178	7720		
7	14.09	1.95			6141		12282	23900		
8	20.20	4.42			15931		31862	140500		
Σ	70.53	9.24	896.98	25.79	-26770		-53540	-174446		

$$H_o = \frac{[8(-174446)-(-53540)9.24]}{-242.4} = 3710 \text{ LBS.} \qquad V_o = 0$$

$$M_o = -[-53540+(2\times 3710\times 9.24)]/16 = -935 \text{ FT. LBS.}$$

TABLE #13

SUMMATION TABLE — LIVE LOAD FOR MAX. + M_S

Point	X	Y	X^2	Y^2	M_R (-)	M_L (-)	M_R+M_L (-)	$(M_R+M_L)Y$ (-)	M_R-M_L (+)	$(M_R-M_L)X$ (+)
1	0.99	0.04				0		0	0	0
2	2.98	0.10	SAME AS TABLE #9	SAME AS TABLE #9		0	SAME AS M_L	0	0	0
3	4.95	0.24				0		0	0	0
4	6.92	0.45				318		143	318	2220
5	9.00	0.79				1291		1020	1291	11620
6	11.30	1.25				3089		3860	3089	34900
7	14.09	1.95				6141		11940	6141	86250
8	20.20	4.42				15931		70400	15931	322000
Σ	70.53	9.24	896.98	25.79		-26770		-87363	+26770	+456970

$$H_o = \frac{[8(-87363)-(-26770)\times 9.24]}{242.4} = 1864 \text{ LBS.} \qquad V_o = \frac{456970}{1793.96} = 254.5 \text{ LBS}$$

$$M_o = -\frac{[-26770+(2\times 1864\times 9.24)]}{16} = 471 \text{ FT. LBS.}$$

TABLE #14

SUMMATION TABLE — LIVE LOAD FOR MAX. - M_S

Point	X	Y	X^2	Y^2	M_R (-)	M_L (-)	M_R+M_L (-)	$(M_R+M_L)Y$ (-)	M_R-M_L (+)	$(M_R-M_L)X$ (+)
1	0.99	0.04			0	0	0	0	0	0
2	2.98	0.10	SAME AS TABLE #9	SAME AS TABLE #9	622	622	1244	124	0	0
3	4.95	0.24			1852	1852	3704	889	0	0
4	6.92	0.45			3697	3390	7087	3185	307	2125
5	9.00	0.79			6294	5015	11309	8845	1279	10490
6	11.30	1.25			9890	6812	16702	20900	3078	38400
7	14.09	1.95			15118	8990	24108	47000	6128	86250
8	20.20	4.42			28717	13762	43479	191800	15955	322000
Σ	70.53	9.24	896.98	25.79	-67190	-40443	-107633	-272743	-26747	-459265

$$H_o = \frac{[8(-272743)-(-107633)9.24]}{-242.4} = 4899 \text{ LBS.} \qquad V_o = \frac{459265}{1793.96} = -265.5 \text{ LBS.}$$

$$M_o = -\frac{[-107633+(2\times 4899\times 9.24)]}{16} = +1076 \text{ FT. LBS.}$$

A summary of the values of H_0, V_0, and M_0, obtained from the summation tables are as follows:

Case I. From Table 9.

$H = 25450$#s; $V = 0$; $M = -453$ ft#s; $e = -0.017$ ft.

Case II. From Tables 9 and 10.

$H = 25450 + 6780 = 32230$#s; $V = 0$; $M = -453 + 575 = 122$ ft#s;
$e = 0.004$ ft.

Case III. From Tables 9 and 11.

$H = 25450 + 3050 = 28500$#s; $V = 0$; $M = -453 + 962 = 509$ ft#s;
$e = 0.018$ ft.

Case IV. From Tables 9 and 12.

$H = 25450 + 3710 = 29160$#s; $V = 0$; $M = -453 - 935 = -1388$ ft#s;
$e = -0.048$ ft.

Case V. From Tables 9 and 13.

$H = 25450 + 1864 = 27314$#s; $V = 254.5$#s; $M = -453-471 = -924$ft#s;
$e = -0.034$ ft.

Case VI. From Tables 9 and 14.

$H = 25450+4899 = 30349$#s; $V = -265.5$#s; $M = -453+1076 = 623$ ft#s;
$e = 0.021$ ft.

Having computed these values, force polygons and their corresponding equilibrium polygons were drawn for all six conditions of loading. The point of application of the line of pressure was now definitely known at the crown, the eccentric distance $e = \frac{M_0}{H_0}$, the sign depending on the sign of M_0. A positive sign meant that the point of application was at a distance "e" above the neutral axis, and a negative sign, below. The equilibrium

polygons were drawn by making parallel transposition of each successive ray of the force polygons to its place between the proper load action lines, beginning in each case at the crown and working toward the springing.

The bending moments, thrusts, shears, and eccentric distances for the six conditions of loading were shown in Tables 15,16,17,18,19, and 20, in which ten points were considered, the crown, the springing, and the center of the eight $\frac{ds}{I}$ constant divisions of the arch axis. Bending moments were calculated from the formula

$$M = m + H_O y + M_O \pm V_O x$$

where m = the bending moment at any point in the cantilever due to external loads, assumed positive when causing compression in the upper fibres.

H_O = the thrust at the crown.

M_O = the bending moment at the crown.

V_O = the shear at the crown.

x and y = the coordinates of any point on the arch axis referred to the crown as the origin, and all to be considered as having a positive sign.

The plus sign of $V_O x$ was used for the left half, and the minus sign for the right half of the arch.

The thrusts and the shears were determined graphically and scaled from the proper force polygon, the thrust being the component of the resultant perpendicular to the arch radius at any point drawn through the true pole. The shear was the component parallel to the arch radius at that point, drawn through the intersection of the force polygon rays with the vertical string polygon. The eccentric distances were equal to the moments divided by their corresponding thrusts.

TABLE #15

NDING MOMENTS, THRUSTS, AND SHEARS.

O; M_o = -453 FT LBS. DEAD LOAD ONLY - L.H. SPAN.

	$H_o y$	x	$V_o x$	M_o	M	Thrust	Ecc. Distance	Shear
16	+ 182250	24.5	0	-450	3550	30200	.184	2600
42	+ 112500			-450	150	27600	.005	2300
95	+49600			-450	-1280	26350	-.049	800
.25	+ 31800			-450	- 62	26000	-.002	450
0.79	+ 20100			-450	+382	25800	+.015	550
045	+ 11450			-450	+ 24	25600	+.001	700
0.24	+ 6100			-450	+283	25550	+.011	750
0.10	+ 2550			-450	+326	25450	+.013	700
0.04	+ 1020			-450	+620	25450	+.024	600
0.00	0			-450	-450	25450	-.017	0

TABLE #16

BENDING MOMENTS, THRUSTS, AND SHEARS.

S; $V_o = 0$; M_o = 122 FT.LBS. D.L.+L.L. ON WHOLE SPAN - R.H. SPAN.

	y	$H_o y$	x	$V_o x$	M_o	M	Thrust	Ecc. Distance	Shear	
	7.16	231000			122	12672	37650	.336	-3200	Change Signs Before Using Table #24
1	4.42	142500			122	1005	34800	.029	-3150	
9	1.95	62800			122	-2626	33400	-.079	-550	
2	1.25	40300			122	-880	33000	-.027	-100	
62	0.79	25450			122	10	32700	.000	-400	
73	0.45	14500			122	-51	32550	-.002	-500	
19	0.24	7750			122	653	32450	-.020	-600	
96	0.10	3220			122	946	32350	+.029	-700	
0	0.04	1290			122	1412	32300	.044	-700	
0	0.00	0			122	122	32230	.004	0	

TABLE #17

NDING MOMENTS, THRUSTS, AND SHEARS

=O ; M_0 = 509 FT. LBS.   DL+LL. FOR MAX. +M_c. L.H. SPAN

	H_0y	x	V_0x	M_0	M	Thrust	Ecc. Distance	Shear
.16	204000			509	11709	33150	.354	3800
.42	126000			509	847	30600	.028	3000
1.95	55500			509	-3411	29500	-.116	900
1.25	35600			509	-2115	29100	-.073	200
0.79	22500			509	-1274	28900	-.044	-100
0.45	12800			509	-1057	28750	-.037	150
0.24	6840			509	130	28650	.005	300
0.10	2850			509	963	28600	.034	450
0.04	1140			509	1649	28500	.058	650
0.00	0			509	509	28500	.018	0

TABLE #18

BENDING MOMENTS, THRUSTS, AND SHEARS.

BS: V_0=O; M_0 = -1388 FT. LBS   DL+LL. FOR MAX. -M_c R.H. SPAN.

	y	H_0y	x	V_0x	M_0	M	Thrust	Ecc. Distance	Shear
71	7.16	208500			-1388	5641	34650	.162	-2550
31	4.42	128700			-1388	-519	31650	-.016	-2300
71	1.95	56900			-1388	-1059	30250	-.035	-600
01	1.25	36400			-1388	511	29850	.017	50
559	0.79	23000			-1388	1033	29500	.036	-400
294	0.46	13100			-1388	418	29250	.014	-700
367	0.24	7000			-1388	245	29200	.008	-950
744	0.10	2920			-1388	-212	29160	-.007	-700
0	0.04	1165			-1388	-223	29160	-.008	-600
0	0.00	0			-1388	-1388	29160	-.048	0

Change Signs Before Using Table #24

TABLE #19

NDING MOMENTS, THRUSTS, AND SHEARS.

254.5 LBS; M_0 = -924 FT. LBS.   D.L. + L.L. FOR MAX. $+M_s$

	$H_0 y$	x	$V_0 x$	M_0	M	Thrust	Ecc. Distance	Shear
	195500	243	6190	-924	7939	32100	.247	3000
2	120500	202	5740	-924	-946	29600	-.034	2500
5	53200	14.09	3580	-924	-3564	28280	-.0126	300
5	34100	11.30	2880	-924	-2168	28000	-.078	0
9	21550	9.00	2290	-924	-1367	27800	- .049	-100
45	12250	6.92	1765	-924	-1275	27600	-.046	-100
24	6550	4.95	1260	-924	-333	27450	-.012	100
10	2730	2.98	760	-924	+170	27380	.006	250
.04	1090	0.99	250	-924	+416	27310	.015	500
00	0	0.00	0	-924	-924	27310	-.034	255
.04	1090	0.99	-250	-924	-84	27320	-.003	-500
.10	2730	2.98	-760	-924	-1350	27400	-.049	-350
.24	6550	4.95	-1260	-924	-2853	27480	-.104	0
.45	12250	6.92	-1766	-924	-5142	27600	-.186	400
0.79	21550	9.00	-2290	-924	-7226	27850	-.259	700
1.35	34100	11.30	-2880	-924	-11006	28100	-.393	1000
1.95	53200	14.09	-3580	-924	-16852	28600	-.590	650
4.42	120500	202	-5740	-924	-27181	30300	-.896	-800
7.16	195300	243	-6190	-924	-29064	33450	-.869	-1000

Change Signs Before Using Table #24

TABLE #20

ENDING MOMENTS, THRUSTS, AND SHEARS

=2655LBS; M_o=623FT.LBS. DL.+L.L. FOR MAX. $-M_s$

	H_oy	x	V_ox	M_o	M	Thrust	Ecc. Distance	Shear	
,	217000	24.3	6450	623	23602	35500	.665	3390	
2	134000	20.2	5360	623	12152	32600	.373	3350	
5	59200	14.09	3730	623	6982	31350	.223	1000	
5	37900	11.30	2990	623	7012	30950	.227	900	
9	23900	8.00	2390	623	5454	30700	.178	900	
5	13620	6.92	1835	623	4784	30550	.156	1250	
24	7280	4.95	1310	623	3846	30550	.126	1300	
10	3035	2.98	790	623	2674	30450	.088	1000	
04	1210	0.99	262	623	2095	30400	.069	850	
00	0	0	0	623	623	30350	.021	-256	
04	1210	0.99	-262	623	1571	30400	.052	-400	Change Signs Before Using Table #24
.10	3035	2.98	-790	623	1094	30450	.036	-500	
.24	7280	4.95	-1310	623	1276	30600	.040	-750	
.45	13620	6.92	-1835	623	1434	30650	.047	-900	
.79	23900	8.00	-2390	623	2865	30750	.093	-950	
25	37900	11.30	-2990	623	4121	31050	.132	-800	
.95	59200	14.09	-3730	623	4663	31300	.149	-1000	
4.42	134000	20.2	-5360	623	17363	32200	.539	-4500	
7.16	217000	243	-6450	623	36923	34450	1.068	-5200	

f a Variation in Temperature of 20^{o}F.

$$H_o = \frac{E}{\frac{ds}{I}} \left(\frac{ctlN}{N\Sigma y^2 - (\Sigma y)^2} \right)$$

$$M_o = \frac{H\Sigma y}{N}$$

E = the modulus of elasticity of concrete = 2,000,000#s per sq.in. = 288,000,000#s/sq.ft.

$\frac{ds}{I}$ = 21.3

c = the coefficient for the expansion of concrete = 0.000006

t = the temperature change = ± 20^{o} F.

l = 51.80 ft. = the span of the arch axis.

N = the number of $\frac{ds}{I}$ constant divisions = 8.

y = 9.24

) = $2(8 \text{x} 25.79 - 9.24^2) = 241.8$

$$H_o = \frac{288,000,000 \text{x} 0.000006 \text{x} 51.8 \text{x} 8 \text{x} 20}{21.3 \text{x} 241.8} = 2780\#\text{s}.$$

$$M_o = - \frac{2780 \text{ x } 9.24}{8} = -3210 \text{ ft\#s}.$$

of arch of one foot

H = ± 7850#s (+) for a rise in temperature,(-)for a fall

M = ± 3210ft#s(-)for a rise in temperature,(+)for a fall

were taken from the drawing by means of a protractor.

TABLE #21

MOMENTS, THRUSTS, SHEARS, AND ECCENTRIC DISTANCES FOR A RISE IN TEMPERATURE OF 20° FAHR.

; $M_o = -3210$ ft#; $M_t = M_o + H_o y$

$H_o y$	M_t	ϕ	$\cos\phi$	Thrust = Tangential Comp. of H_o	$\sin\phi$	Shear = Normal Comp of H_o	Eccentric Distance
9900	16690	37°-0'	.800	2225	.600	1668	7.500
2 300	9090	28°-15'	.881	2450	.472	1321	3.710
5430	2270	16°-05'	.961	2650	.276	767	0.856
3480	270	11°-46'	.979	2690	.204	568	0.101
2200	-1010	9°-23'	.987	2745	.163	453	-0.368
1250	-1960	6°-55'	.992	2755	.114	317	-0.712
668	-2542	5°-0'	.996	2770	.087	242	-0.918
278	-2932	3°-10'	.998	2773	.055	153	-01.058
111	-3099	0°-52'	.999	2776	.016	42	-1.117
0	-3210	0°-0'	1.000	2778	.000	0	-1.153

ening
rust throughout the arch which produced an average stress
icrete equal to f_c lbs. per square inch would shorten the
an amount equal to $\frac{f_c l}{E}$, if unrestrained. This action
horizontal reactions in the same manner as a lowering
iperature. For this thrust,

$$\frac{-I \quad f_c l N}{ds\,2(N\Sigma y^2-(\ y)^2)} \qquad \text{and } M_0 = \frac{-H_0 \Sigma y}{N}$$

verage compressive stress f_c at any section was the
that section, divided by the area of the transformed
the arch ring; the value of f_c that was used in the
s the average for the arch.
ormula for the area of the transformed section is
= bh + n(A + A') or bh + 2nA for symmetrical sections
"h" was scaled from the drawing ; "b" was taken as 1'0".
ng the average value of f_c for the arch, it was neces-
to consider the stresses at the crown, at the springing,
center of the sixth $\frac{ds}{I}$ constant division. The thrust
shortening was negative, and therefore decreased the av-
·essive stress in the arch due to the dead load, live
emperature changes. Before the value f_c could be ob-
was necessary to know the value of H_0 due to rib
Because of this, a value of H_0 due to rib shorten-
umed, after which the average stress f_c in the arch
ned, and the value was computed. No revision of com-
s made, if the computed value agreed within ten per
assumed value.

ases were considered: first, that due to the dead load, , rise in temperature, and rib shortening; and, second, to dead load, live load, fall in temperature, and rib g.

values of the bending moments, thrusts, and shears were by multiplying those due to temperature change by the H_0 for rib shortening to H_0 for temperature change. The e all opposite for temperature rise. The results were Table No. 22.

Case I

oad, Temperature Rise and Rib Shortening.

H_0 for rib shortening = -3000#s.

trial.

c at crown $= \dfrac{25450+2780-3000}{1\text{x}1.00+.21} = 20850\#\text{s}/_{\text{sq.ft.}}$

c at point 6 $= \dfrac{26000+.979(2780-3000)}{1\text{x}1.11+.21} = 19500\#\text{s}/_{\text{sq.ft.}}$

h was taken from Cuwe 1. The distance for point 6 he crown along the arch axis was 11.85 ft.

at springing $= \dfrac{30200+.8(-220)}{1\text{x}2.45+.21} = 11300\#\text{s}/_{\text{sq.ft.}}$

for arch $= \dfrac{20850+19500+11300}{3} = 17217\#\text{s}/_{\text{sq.ft.}}$

$\dfrac{17217\text{x}51.8\text{x}8}{21.3\text{x}241.8} = -1388\#\text{s}$ (Too small)

trial. Assume $H_0 = -1400\#\text{s}$.

at crown $= \dfrac{25450+1380}{1.21} = 22{,}100\#\text{s}/_{\text{sq.ft.}}$

at point 6 $= \dfrac{25450+.979(1380)}{1.32} = 20{,}700\#\text{s}/_{\text{sq.ft.}}$

t springing $= \frac{30200+.8(1380)}{2.66}$ $= 11750$#s/sq.ft

or the arch $= \frac{22100+20700+11750}{3}$ $= 18183$#s/sq.ft

$\frac{.83x51.8x8}{1.3x241.8}$ $= -1460$#s; $M_o = +\frac{1460x9.24}{8} = 1688$ft#s.

value for H_o is within ten per cent of the assumed

e value for H_o will be taken as final

, Live Load, Temperature Rise,and Rib Shortening.

alue of H_o = -1700#s.

crown $= \frac{32230+(2780-1700)}{1.21}$ $= 27500$#s/sq.ft.

point 6 $= \frac{33000+.979x1080}{1.32}$ $= 25800$#s/sq.ft.

springing $= \frac{37650+.8x1080}{2.66}$ $= 14480$#s/sq.ft.

r arch $= \frac{27500+25800+14480}{3}$ $= 22593$#s/sq.ft.

$= -\frac{22593x51.8x8}{21.3x241.8}$ $= -1815$#s; $M_o =$

$- \frac{-1815x9.24}{8}$ $= 2100$ ft#s.

Live Load for Max. $+M_c$, Temperature Rise, Rib

-1700#s.

crown $= \frac{28500+(2780-1700)}{1.21}$ $= 24400$#s/sq.ft.

point 6 $= \frac{29100+.979x1080}{1.32}$ $= 22900$#s/sq.ft.

springing $= \frac{33150+.800x1080}{2.66}$ $= 12800$#s/sq.ft.

arch $= \frac{24400+22900+12800}{3}$ $= 20033$#s/sq.ft.

$\frac{51.8x8}{41.8}$ $= -1615$#s; $M_o = -\frac{-1615x9.24}{8} = 1865$ ft#s.

ad, Live Load for Max. $-M_c$, Temperature Rise, Rib

ing.

$H_0 = -1680\#s.$

at crown $= \frac{29160+(2780-1680)}{1.21} = 25000\#s/_{sq.ft.}$

at point 6 $= \frac{29850+.979x1100}{1.32} = 23400\#s/_{sq.ft.}$

at springing $= \frac{34650+.800x1100}{2.66} = 13350\#s/_{sq.ft.}$

for arch $= \frac{25000+23400+13350}{3} = 20550\#s/_{sq.ft.}$

$\frac{550x51.8x8}{1.3x241.8} = -1650\#s; \; M_0 = -\frac{-1650x9.24}{8} = 1905 \text{ ft}\#s.$

ad, Live Load for Max. $+M_s$, Temperature Rise, Rib

ing.

$H_0 = -1480\#s.$

l.springing $= \frac{32100+.800x1300}{2.66} = 12440\#s/_{sq.ft.}$

l.point 6 $= \frac{28000+.979x1300}{1.32} = 22200\#s/_{sq.ft.}$

at crown $= \frac{27314+ 1300}{1.21} = 23700\#s/_{sq.ft.}$

r.point 6 $= \frac{28100+.979x1300}{1.32} = 22200\#s/_{sq.ft.}$

r.springing $= \frac{33450+.800x1300}{2.66} = 12950\#s/_{sq.ft.}$

for arch $= \frac{12440+22200+23700+22200+12950}{5} = 18698\#s/ft^2$

$\frac{8698x51.8x8}{21.3x241.8} = -1500\#s; \; M_0 = - \frac{-1500x9.24}{8} = 1736 \text{ ft}\#s.$

oad, Live Load for Max.-M_s, Temperature Rise, Rib

ning.

H_o = -1680#s.

$_c$ l.springing $= \frac{35500+.800x1100}{2.66} = 13650$#s/$_{sq.ft.}$

$_c$ l.point 6 $= \frac{30950+.979x1100}{1.32} = 24300$#s/$_{sq.ft.}$

$_c$ at crown $= \frac{30350+1100}{1.21} = 26000$#s/$_{sq.ft.}$

$_c$ r.point 6 $= \frac{31050+.979x1100}{1.32} = 24300$#s/$_{sq.ft.}$

$_c$ r.springing $= \frac{34550+.800x1100}{2.66} = 13300$#s/$_{sq.ft.}$

$_c$ for arch $= \frac{13650+24300+26000+24300+13300}{5} = 20310$#s/$ft^2$

$\frac{20310x51.8x8}{21.3x241.8} = -1632$#s; $M_o = - -\frac{1632x9.24}{8} = 1885$ ft#s.

Case II

same sort of calculations were used for Case II as for

irst, a value of H_o was assumed; then an average f_c

ined; and finally, H_o was computed. If the computed

within ten per cent of the assumed value, the computed

considered within the required limits of accuracy.

ad, Temperature Fall, and Rib Shortening.

H_o = -1320#s.

for arch = 14800#s/$_{sq.ft.}$

$\frac{4800x51.8x8}{1.3x241.8} = -1190$#s; $M_o = --\frac{1190x9.24}{8} = 1375$ ft#s.

d Load, Live Load, Temperature Fall, Rib Shortening.

ume H_o = -1520#s.

, f_c for arch = 19217#s/sq.ft.

$\frac{-19217x51.8x8}{21.3x241.8} = -1545$#s; $M_o = - \frac{-1545x9.24}{8} = 1782$ ft#s.

l Load, Live Load for Max. $+M_c$, Temperature Fall, Rib

:tening.

ume H_o = -1420#s.

, f_c for arch = 16733#s/sq.ft.

: $\frac{-16733x51.8x8}{21.3x241.8} = -1348$#s; $M_o = - \frac{-1348x9.24}{8} = 1555$ ft#s.

l Load, Live Load for Max. $-M_c$, Temperature Fall, Rib

·tening.

ume H_o = -1320#s.

f_c for arch = 17360#s/sq.ft.

$\frac{-17360x51.8x8}{21.3x241.8} = -1395$#s; $M_o = - \frac{-1395x9.24}{8} = 1610$ ft#s.

Load, Live Load for Max. $+M_s$, Temperature Fall,

Shortening.

me H_o = -1220#s.

f_c for arch = 15608#s/sq.ft.

$\frac{-15608x51.8x8}{21.3x241.8} = -1255$#s; $M_o = - \frac{-1255x9.24}{8} = 1450$ ft#s.

Load, Live Load for Max. $-M_s$, Temperature Fall,

Shortening.

ne H_o = -1320#s.

f_c for arch = 17290#s/sq.ft.

$\frac{-17290x51.8x8}{21.3x241.8} = -1386$#s; $M_o = - \frac{-1386x9.24}{8} = 1600$ ft#s.

TABLE #22

NDING MOMENTS, THRUSTS, & SHEARS DUE TO RIB SHORTENING

	Case I (a)			Case I (b)		

	o = −1460 #s; M_o = 1688 ft#s.
	atio = $\frac{H_o\text{ (Rib Shortening)}}{H_o\text{ (Temperature)}}$ = −.525
				H_o = −1815 #s M_o = 2100 ft#s.
				Ratio = $\frac{H_o\text{ (Rib Shortening)}}{H_o\text{ (Temperature)}}$ = −.655		

t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
g	−8740	−1167	−874	−10870	−1451	−1087
	−4760	−1285	−689	−5926	−1598	−857
	−1190	−1390	−401	−1482	−1730	−500
	−141	−1410	−298	−176	−1755	−371
	+530	−1440	−238	660	−1793	−296
	+1028	−1446	−166	1279	−1796	−207
	+1334	−1451	−127	1660	−1809	−158
	+1534	−1452	−80	1911	−1810	−100
	+1622	−1453	−22	2020	−1811	−27
n	+1683	−1457	−0	2095	−1813	0

	Case I (c)			Case I (d)		

	o = −1615 #s; M_o = 1865 ft#s.
	atio = $\frac{H_o\text{ (Rib Shortening)}}{H_o\text{ (Temperature)}}$ = −.581
				H_o = −1650 #s; M_o = 1908 ft#s
				Ratio = $\frac{H_o\text{ (Rib Shortening)}}{H_o\text{ (Temperature)}}$ = −.594		

t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
g	−9680	−1292	−969	−9890	−1318	−968
	−5280	−1422	−763	−5390	−1450	−778
	−1319	−1539	−446	−1343	−1570	−455
	−157	−1564	−330	−160	−1594	−337
	587	−1595	−265	598	−1626	−268
	1140	−1600	−184	1160	−1632	−188
	1480	−1610	−141	1509	−1640	−143
	1703	−1611	−89	1738	−1642	−91
	1800	−1612	−24	1832	−1644	−25
n	1865	−1614	−0	1901	−1648	−0

TABLE #22 (CONTINUED)

	Case I (e)			Case I (f)		
	H_o - 1500 #s ; M_o = 1736 ft#s			H_o = -1632 #s ; M_o = 1885 ft#s.		
	Ratio = $\frac{H_o(\text{Rib Shortening})}{H_o(\text{Temperature})}$ = -.540			Ratio = $\frac{H_o(\text{Rib Shortening})}{H_o(\text{Temperature}}$ = -.587		
t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
ng	- 9000	- 1200	- 900	- 9780	- 1306	- 978
	- 4900	- 1321	- 709	- 5330	- 1438	- 770
	- 1224	- 1429	- 414	- 1331	- 1555	- 450
	- 146	- 1450	- 306	- 159	- 1579	- 333
	545	- 1481	- 245	683	- 1611	- 266
	1058	- 1486	- 171	1150	- 1617	- 186
	1373	- 1495	- 130	1492	- 1625	- 142
	1588	- 1496	- 83	1722	- 1626	- 90
	1670	- 1498	- 23	1816	- 1628	- 25
m	1783	- 1500	0	1881	- 1631	- 0

	Case II (a)			Case II (b)		
	H_o = -1190 #s ; M_o = 1375 ft#s.			H_o = -1545 #s ; M_o = 1782 ft#s.		
	Ratio - $\frac{H_o(\text{Rib Shortening})}{H_o(\text{Temperature})}$ = -.429			Ratio = $\frac{H_o(\text{Rib Shortening})}{H_o(\text{Temperature})}$ = -.556		
t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
ing	- 7140	- 953	- 714	- 9290	- 1238	- 929
	- 3890	- 1049	- 562	- 5060	- 1362	- 730
	- 972	- 1134	- 329	- 1262	- 1473	- 427
	- 116	- 1150	- 243	- 150	- 1496	- 316
	433	- 1176	- 194	562	- 1529	- 252
	840	- 1179	- 136	1090	- 1532	- 176
	1089	- 1184	- 104	1415	- 1540	- 135
	1254	- 1185	- 65	1630	- 1541	- 85
	1325	- 1187	- 18	1724	- 1543	- 23
wn	1373	- 1190	- 0	1787	- 1546	0

TABLE #22 (CONTINUED)

	Case II (c)			Case II (d)		
	$H_o = -1348^{\#s}$; $M_o = 1555$ FT.LBS. $\text{RATIO} = \frac{H_o(\text{RIB SHORTENING})}{H_o(\text{TEMPERTURE})} = -.484$			$H_o = -1395^{\#s}$; $M_o = 1610^{ft\#s}$. $\text{RATIO} = \frac{H_o(\text{RIB SHORTENING})}{H_o(\text{TEMPERATURE})} = -.502$		
t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
ng	-8060	-1066	-806	-8360	-1116	-836
	-4390	-1192	-634	-4560	-1228	-658
	-1095	-1280	-371	-1138	-1330	-385
	-130	-1300	-274	-135	-1350	-285
	488	-1327	-218	507	-1378	-227
	948	-1330	-153	984	-1381	-159
	1230	-1339	-117	1277	-1389	-121
	1415	-1340	-74	1470	-1390	-77
	1496	-1342	-20	1552	-1392	-21
vn	1550	-1344	0	1610	-1395	0

	Case II (e)			Case II (f)		
	$H_o = -1255^{\#s}$; $M_o = 1450^{ft\#s}$ $\text{RATIO} = \frac{H_o(\text{RIB SHORTENING})}{H_o(\text{TEMPERATURE})} = -.451$			$H_o = -1386^{\#s}$; $M_o = 1600^{ft\#s}$ $\text{RATIO} = \frac{H_o(\text{RIB SHORTENING})}{H_o(\text{TEMPERATURE}} = -.498$		
t	Bending Moment	Thrust	Shear	Bending Moment	Thrust	Shear
ing	-7510	-1003	-751	-8300	-1108	-830
	-4100	-1105	-593	-4520	-1209	-663
	-1024	-1194	-346	-1130	-1320	-382
	-122	-1222	-256	-134	-1339	-283
	455	-1238	-204	502	-1365	-225
	884	-1241	-143	975	-1370	-158
	1148	-1249	-109	1266	-1378	-120
	1321	-1250	-69	1458	-1379	-76
	1397	-1251	-19	1540	-1381	-21
wn	1448	-1253	0	1597	-1383	0

o. 23 contained the combination of bending moments
lue to the six conditions of loading, a rise or fall
re, and rib shortening. By an inspection of the
polygons for the fifth and sixth conditions of load-
noticed that the bending moments were of opposite
esponding points on the opposite sides of the arch.
ositive bending moments with the corresponding
given in Columns 5 and 7, while all of the nega-
moments with corresponding thrusts were given in
d 8. Values in Columns 9 and 16 were taken from
nd values in Columns 10 to 22, inclusive, were taken
o. 22. In the determination of the values for Col-
24, the three causes of stress were always kept in
ondition of loading, the rise or fall in temperature,
rtening.

mns 1 to 8, inclusive, contain the resulting moments
due to the six conditions of loading; Columns 9 and
d moments and thrusts due to rise or fall in temper-
Columns 10 to 15 inclusive and 17-22 inclusive con-
moments and thrusts due to rib shortening. In order
e a moment value for 23, a moment was selected from
irst cause, another from the third cause, and the sum
ments were added to the moment found in Column 9.
e a moment value for Column 24, a moment was selected
ns 1-8 inclusive, another from Columns 17-22 inclusive,
n of these were added to the value found in Column 16.
corresponding to each moment was treated in the same
tive moments meant compression at the extrados, and

Spring	30200	5550	37650	12672	33150	11709	34650	5641	32100	7939	33450	-29064	34550	36923
8	27600	150	34800	1005	30600	847	31650	-519	29900	-946	30300	-27181	32200	17363
7	26350	-1280	33400	-2626	29500	-3411	30250	-1059	28280	-3546	28600	-16852	31350	6982
6	26000	-62	33000	-880	29100	-2115	29850	511	28000	-2168	28100	-11006	30950	7012
5	25500	382	32700	10	28900	-1274	29500	1053	27870	-1367	27850	-7226	30070	5454
4	25600	24	32550	-57	28870	-1057	29250	418	27600	-1275	27600	-5112	30560	4786
3	25500	283	32450	653	28650	130	29200	245	27450	-333	27480	-2853	30560	3846
2	25450	326	32350	946	28600	963	29160	-212	27380	170	27400	-1350	30450	2674
1	25400	620	32300	1142	28500	1649	29160	-223	27310	416	27320	-84	30400	2096
Crown	25400	-450	32230	122	28500	509	29160	-1388	—	—	27310	-924	30300	623

	8		9		10		11		12		13		14	
	D.L. + L.L. for Max M_s - B.M.		Temperature Rise		Rib Shortening Case I									
					(a)		(b)		(c)		(d)		(e)	
Point	Thrust	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment
Spring	35500	-23600	2225	16690	-1167	-8740	-1451	-10870	-1292	-9680	-1318	-9890	-1200	-9000
8	32600	-12152	2450	9090	-1285	-4760	-1518	-5925	-1422	-5280	-1450	-5390	-1321	-4900
7	31300	-4663	2650	2270	-1390	-1190	-1730	-1482	-1539	-1319	-1570	-1343	-1429	-1224
6	31050	-4121	2690	270	-1410	-141	-1755	-176	-1564	-157	-1594	-160	-1450	-146
5	30750	-2865	2745	-1010	-1440	530	-1793	660	-1595	587	-1626	598	-1481	545
4	30650	-1434	2755	-1960	-1445	1028	-1796	1279	-1600	1140	-1632	1160	-1485	1058
3	30600	-1226	2770	-2252	-1451	1334	-1809	1660	-1610	1480	-1640	1509	-1495	1373
2	30450	-1094	2773	-2932	-1452	1534	-1810	1911	-1611	1703	-1642	1738	-1496	1580
1	30400	-1571	2776	-3099	-1453	1622	-1811	2020	-1612	1800	-1644	1832	-1498	1670
Cr	—	—	2780	-3210	-1457	1683	-1813	2095	-1614	1865	-1648	1901	-1500	1733

TABLE #23 (CONTINUED)

15		16		17		18	
tening · Case I		Temperature Fall		Rib Shortening · Case II			
(f)				(a)		(b)	
ust	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment
06	- 9780	-2225	-16690	-953	-7140	-1238	- 9230
38	- 6330	-2450	- 9090	-1049	- 3890	-1362	- 5060
55	- 1331	-2650	- 2270	-1134	- 972	-1473	- 1262
79	- 159	-2690	- 270	-1150	- 116	-1496	- 150
91	593	-2745	1010	-1176	433	-1528	562
17	1150	-2755	1960	-1179	840	-1532	1090
25	1492	-2720	2542	-1184	1089	-1540	1415
26	1720	-2773	2952	-1185	1254	-1641	1630
28	1816	-2776	3099	-1187	1325	-1543	1724
1	1881	-2780	3210	-1190	1373	-1546	1787

19		20		21		22	
		Rib Shortening - Case II					
		(d)		(e)		(f)	
	Moment	Thrust	Moment	Thrust	Moment	Thrust	Moment
	- 8860	-1115	- 8360	-1003	- 7510	-1108	- 8300
	- 4390	-1228	- 4560	-1105	- 4100	-1209	- 4620
	- 1035	-1330	- 1138	-1194	- 1024	-1320	- 1130
	- 136	-1350	- 135	-1222	- 122	-1339	- 134
	448	-1378	507	-1238	455	-1366	502
	948	-1381	984	-1241	884	-1370	975
	1230	-1389	1277	-1249	1148	-1378	1266
	1476	-1390	1470	-1250	1321	-1379	1458
	1486	-1392	1552	-1251	1397	-1380	1540
	1550	-1395	1610	-1253	1448	-1383	1597

TABLE #23 (CONTINUED)

23			24		
perature Rise and Rib Shortening Columns 1, 2, 3, 4, 5, 6, 7, or 8.			Temperature Fall and Rib Shortening plus Columns 1, 2, 3, 4, 5, 6, 7, or 8.		
ent	Thrust	Ecc. Distance	Moment	Thrust	Ecc. Distance
22	37650				
59	34208				
89	33158		-55044	29987	- 1.84
73	35608	1.26	-49580	32037	
			-14669	27838	
5	35965		-15096	25788	
7	31765		-41331	26488	- 1.56
3	33365				
			-6158	29277	
2	32610		-8195	29177	
1	31550		20384	24477	- 0.83
7	27280		-2535	24914	
0	31130		11426	23914	- 0.48
1	32230		4541	26864	
3	30805		-3654	23376	-0.24
4	32005	+.15	-1293	26476	
			-2062	23313	-0.09
	31860	+.12	1616	26362	
			+1104	23170	
	31869	+.08	2731	26290	+.10
	33671		+3272	23 086	+.140
	31771		3188	26286	+.120
	29823		4600	24841	+.180
	31723		3252	26081	
	31676		3609	24834	
	33553		4073	22984	+.180

$T + \frac{6}{d} M$

$37650 + 6/2.45 \times 20622 = 37650 + 50550 = 88200$
$9580 + 2.45 \times 32037 = 49580 + 78510 = 128080$
$55044 + 2.45 \times 29987 = 55044 + 73400 = \underline{128444}$ Max.+.

$7838 + 6/1.68 \times 14669 = 27838 + 52400 = 80238$
$3365 + 3.57 \times 21693 = 33365 + 77500 = 110865$
$6488 + 3.57 \times 41331 = 26488 + 147400 = \underline{173888}$ Max.+.

$9177 + 6/1.17 \times 8195 = 29177 + 41990 = 71167$
$2610 + 5.13 \times 8242 = 32610 + 42240 = 74850$
$4477 + 5.13 \times 20384 = 24477 + 104200 = \underline{128677}$ Max.+.

$2230 + 6/1.05 \times 7141 = 32230 + 40700 = 72930$
$3914 + 5.71 \times 11426 = 23914 + 65300 = \underline{89214}$ Max.+.

$476 + 5.95 \times 1293 = 26476 + 7700 = 34176$
$3576 + 5.95 \times 5654 = 23576 + 33100 = 56676$
$005 + 5.95 \times 4974 = 32005 + 29600 = \underline{61605}$ Max.+.

$3313 + 6 \times 2062 = 23313 + 12380 = 35693$
$363 + 6 \times 1616 = 26363 + 9700 = 36063$
$860 + 6 \times 3852 = 31860 + 23100 = \underline{54960}$ Max.+.

$290 + 6 \times 2638 = 26290 + 15728 = 41918$
$969 + 6 \times 2731 = 31869 + 16386 = \underline{48255}$ Max.+.

$23 + 6 \times 618 = 31723 + 3708 = 35431$
$341 + 6 \times 4600 = 24841 + 27600 = \underline{52441}$ Max.+.
$81 + 6 \times 3252 = 26081 + 19512 = 45593$

$1 + 6 \times 1276 = 31771 + 7656 = 39427$
$86 + 6 \times 3188 = 26286 + 19128 = \underline{45414}$ Max.+.

$3 + 6 \times 1403 = 33553 + 8418 = 41971$
$4 + 6 \times 4073 = 22984 + 24438 = \underline{47422}$ Max.+.

$$T - \frac{6}{d} M$$

$\begin{cases} 34200 - 2.45 \times 19659 = 34200 - 48200 = -14000 \\ 35608 - 2.45 \times 44873 = 35608 - 109800 = \underline{\underline{-74200}} \end{cases}$ Max.(-).

$\begin{cases} 33365 - 3.57 \times 21693 = 33365 - 77500 = -44135 \\ 26488 - 3.57 \times 41331 = 26488 - 147400 = \underline{\underline{-120912}} \end{cases}$ Max(-)

$\begin{cases} 29177 - 5.13 \times 8195 = 29177 - 41990 = -12813 \\ 24477 - 5.13 \times 20384 = -104200 + 24477 = -79723 \end{cases}$ Max(-).

$\begin{cases} 32230 - 5.71 \times 7141 = 32230 - 40700 = -8470 \\ 23914 - 5.71 \times 11426 = 23914 - 65300 = -41386 \end{cases}$ Max.(-).

$\{ 23576 - 5.95 \times 5654 = 23576 - 33100 = -9524$ Max(-)

$\begin{cases} 23313 - 6 \times 2062 = 23313 - 23100 = 213 \\ 31860 - 6 \times 3852 = 31860 - 12380 = 19486 \end{cases}$ Max(-)

$\begin{cases} 26290 - 6 \times 2731 = 26290 - 16386 = 9904 \\ 23170 - 6 \times 1104 = 23170 - 6624 = 16546 \end{cases}$ Max(-)

$23086 - 6 \times 3272 = 23086 - 19632 = 3454$ Max(-)
$26286 - 6 \times 3188 = 26286 - 19128 = 7158$

$24841 - 6 \times 4600 = 24841 - 27600 = -2759$ Max(-)

$22984 - 6 \times 4073 = 22984 - 24438 = -1454$ (Max(-)
$24834 - 6 \times 3609 = 24834 - 21654 = 3180$

tension at the intrados.

In order to determine which moment and thrust would give the greatest compressive stress in the concrete, it was necessary to apply the criterion $T + \frac{6M}{d}$ (all "M" were considered positive in sign), where T was the thrust, d = h, and M was the moment. It was found that the criterion gave highest results for combinations of greatest positive moment and greatest thrust; hence, only those combinations were entered in Column 23. The combination of moment giving the greatest value for $T+\frac{6M}{d}$ was underscored, as it was the one which gave the greatest maximum stress. The criterion $T - \frac{6M}{d}$ (all "M" were considered positive in sign) was used to determine which combination of moment and thrust gave the greatest tensile stress in the steel. The combination of moment and thrust which gave the greatest negative value of $T - \frac{6M}{d}$ was underscored. Only the combinations of greatest negative moments and smallest thrusts were entered in Column 24, since they gave the highest value for the criterion $T - \frac{6M}{d}$. If there were no negative moments, the lowest positive moments were entered in Column 24.

The eccentric distances in Columns 23 and 24 were the bending moments divided by the thrusts.

Table No. 24 combined the shears due to the various conditions of loading with those due to a rise or fall in temperature and rib shortening. The maximum positive shear was obtained by adding to the shear due to rise in temperature and rib shortening (Col.9), the greatest positive shear in Columns 1 to 8 inclusive. The maximum negative shear was obtained by

8	2300	3150	3000	2300	2500	800	4500	3350
7	800	550	900	600	800	-650	1950	1000
6	450	100	200	-50	0	-1000	900	800
5	550	400	-100	400	-100	-700	950	900
4	700	500	160	700	-100	-400	1250	900
3	750	600	300	950	100	0	1300	750
2	700	700	450	700	350	250	1000	500
1	600	700	650	600	500	500	850	400
Crown	0	0	0	0	255	—	—	266

	9	10	11	12	13	14	15
Point	Case I Temp. Rise & Rib Shortening	Case II Temp. Fall & Rib Shortening	Cross Section (□')	Maximum Shear +	Maximum Shear −	Max. Unit Shear (#/□')	Max. Unit Shear (#/□")
Spring	794	- 2597	2.45	5994	- 1597	2440	17.6
8	622	- 2041	1.68	5122	- 1241	3025	21.0
7	367	- 1194	1.17	2717	- 1844	1980	13.7
6	270	- 884	1.05	1170	- 1884	1795	12.5
5	215	- 705	1.01	1165	- 1405	1389	9.6
4	151	- 493	1.00	1401	- 893	1400	9.7
3	115	- 377	1.00	1415	- 377	1415	9.8
2	73	- 238	1.00	1073	+ 12	1073	7.4
1	20	- 65	1.00	870	+ 335	870	6.0
Crown	0	0	1.00	266	+ 255	266	1.9

adding to the shear due to the fall in temperature and rib shortening (Column 10), the greatest negative shear in Columns 1 to 8 inclusive.

Table No. 25 furnished a means to compute stresses due to bending moments and eccentric distances. Two cases which must be considered were: first, when the fibre stress was wholly compressive (the line of pressure lying within the kern), and, second, when there was some tension at the section (the line of pressure lying without the kern). In order to determine the case under which the section came, the Table on page 101 of Turneaure and Maurer's "Principles of Reinforced Concrete Construction" was used. After the quantities "p" and $\frac{e}{h}$ were determined, values of $M/_{bh^2 f_c}$ were obtained from Plates 12 and 13 of Turneaure and Maurer, depending upon the case. Values of f_c were obtained by dividing the values of $M/_{bh^2}$ by the quantities $M/_{bh^2 f_c}$. The value for h for the second case was obtained from Plates 11 and 12 of Turneaure and Maurer. The tension in the steel was determined from the formula

$$f_s = nf_c(\frac{d}{kh} - 1)$$

where f_c = the stress in the steel

n = the ratio of E_s/E_c.

f_c = the stress in the concrete

d = the distance from the compressive face to the plane of the steel

k = the ratio of the depth of the neutral axis of the section below the top to "d"

h = the radial depth of the arch

TABLE #25

MAXIMUM STRESSES

$A = 0.007^{d}$; $d' = 3'' = 0.25'$

								Maximum Compression in Concrete, Case I or Case II					Maximum Tension in Steel Case II								
Point	h	$p = \frac{A}{bh}$	h^2	e	e/h		$\frac{M}{b^2h^2f_c}$	M	$\frac{M}{bh^2}$	f_c (#/□")	Where & When it occurs	f_c (#/□")	M	W EW LO	K Case II	$d = (h-d')$	Kh	d/Kh	$d/Kh - 1$	nf_c	f_s (#/□")
Sp	2.45	.003	6.00	-1.84 1.26	.750 .514	II II	.153 .148	-55044	9200	60000	-F	417	44873	-R	.618	2.20	1.51	1.45	.45	6250	2810
8	1.68	.004	2.82	-1.56	.928	II	.157	-44331	15700	100000	-F	695	44331		.420	1.43	.705	2.03	1.03	10400	10700
7	1.17	.006	1.37	-0.83	.708	II	.152	-20384	14850	97500	-F	678	-20384		.462	.92	.540	1.70	.70	10150	7100
6	1.05	.007	1.10	-0.48	.457	II	.147	-11426	10400	70800	-F	492	-11426		.588	.85	.616	1.38	.38	7370	2800
5	1.01	.007	1.02	-0.24 0.15	.237 .148	II I	.126 .099	+4974	4880	49200	R	342	-5664	-F	.900	.77	.778	0.99	.01	5130	-51.3
4	1.00	.007	1.00	+0.12 +0.09	.20 .09	I I	.088 .074	-2062	2062	27900	F	194									
3	1.00	.007	1.00	0.10 0.08	.10 .08	I I	.079 .068	2638	2638	38700	-R	269									
2	1.00	.007	1.00	0.14 0.12	.14 .12	I I	.092 .088	3188	3188	36300	-F	282									
1	1.00	.007	1.00	0.18	.18	II	.105	4600	4600	43800	-F	305									
Crown	1.00	.007	1.00	0.18	.18	I	.105	4073	4073	39700	-F	276									

CHAPTER V. DESIGN OF THE ABUTMENTS

The forces acting on the foundation consist of 1) the thrust of the arch, 2) the weight of the foundation, 3) the weight of the earth, pavement, and the live load, and 4) the lateral earth pressure. Resultant lines of pressure were drawn for all possible conditions of loading to determine the maximum pressure on the foundation. The following are the conditions of loading which were assumed: 1) dead load plus live load for Maximum $+M_c$ or Maximum $-M_c$ on the span, or dead load plus live load for Maximum $+M_s$ or Maximum $-M_s$ on the portion of the span farthest from the abutment; dead load plus live load for Maximum $+M_c$ or Maximum $-M_c$ on the span, plus the live load on the abutment or the dead load plus the live load for Maximum $+M_s$ or Maximum $-M_s$ on the portion of the span nearest the abutment; and 3) dead load plus live load on the whole span.

The magnitude and point of application of the thrusts were given in Tables 15, 16, 17, 18, 19, or 20 of the arch design, while the direction of the thrust was given by the force polygons.

The weight of the foundation was the effective dead weight, in which calculations the buoyant effect of the water was taken into account.

The lateral earth pressure is given by Rankine's formulae, which are based on the assumptions that the earth is composed of granular homogeneous particles without cohesion, held only by the internal friction developed between them, and that the mass of earth extends indefinitely.

The weight of the earth, pavement, and the live load were replaced by an equivalent surcharge of earth.

First an abutment was assumed, and drawn to scale on Sheet #3 of the bridge design. The top of the abutment was drawn tangent to the extrados at the springing, and the equivalent depth of fill and live load surcharge was shown. The vertical loads, their centers of gravity, their magnitude, and the line of action of their resultant was computed for loading I. The amount and location of the horizontal earth pressure was computed, and the resultant of the horizontal and vertical forces were drawn to scale along its line of action. The thrusts were determined by combining vectorially the thrusts due to the loading conditions with that due to the temperature change. The resultants, R_I, were drawn by combining the resultant thrusts with the vertical load resultant. The force triangles then consisted of for condition I, the vertical loads (drawn parallel to the resultant of the vertical loads and the horizontal earth pressures), and the final resultant, R_I (drawn through the point where the thrust due to the loading condition pierced the resultant of the vertical loads and the horizontal earth pressures).

By the same methods, the resultants and stress diagrams of conditions II and III were computed and drawn in. Since all of

the lines of pressure lay within the middle third, and since the bearing power of the soil was not exceeded, the first assumed abutment was adopted as final.

If the lines of pressure had fallen outside the middle third, it would have been necessary to redesign the abutment.

Rankine's Earth Pressure Formulae

The lateral earth pressure = $P = C_p w(H^2 - h^2)$

where

P = the resultant earth pressure in pounds on a vertical surface for a horizontal length of one foot.

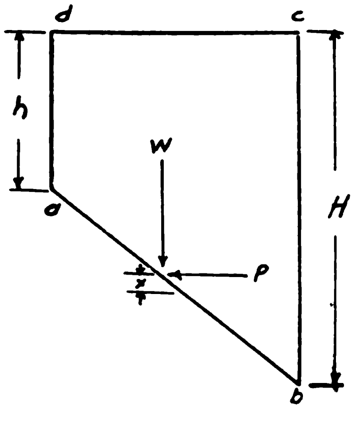
H = the depth of earth above the bottom of surface under consideration

h = height of surcharge

C_p = a constant depending on the angle of inclination of the earth behind the wall. For the conditions of this design, $C_p = 1/6$ or .17, where $\phi = 30^\circ$ for dry sand

The pressure was assumed to act, for vertical faces, parallel to the slope of the earth, and at a point whose distance below the center of the surface is given by the equation

$$x = \frac{(H - h)^2}{6(H + h)}$$



The pressure on an inclined surface is equal to the resultant of P, the pressure on the vertical projection of the surface, and W, the weight of the earth pressure "abcd" one foot in length.

nd minimum pressures were given by the following

x $= \frac{N}{a}(1+\frac{6e}{a})$

n $= \frac{N}{a}(1-\frac{6e}{a})$

e $= \frac{N}{a}$

= the unit pressure in pounds per square foot

= the component of R, the resultant normal to the base

= the eccentricity of the resultant R

= the length of base in feet

bearing power of the soil about Madison was taken

ne-half tons per square foot.

Calculations

ent depth of earth (scaled) = 7.50 ft.

" " " $\frac{150}{120} \times \frac{6\frac{3}{4}}{12}$ = 0.70 ft.

Equivalent depth of earth = 8.20 ft.

harge - equivalent depth of earth = $\frac{125}{120}$ = 1.04 ft

First Assumed Abutment.

Condition of Loading I

Volume(ft^3)	Wt/ft^3	Wt(#)	c.g. (ft.)	M (ft#s)
x2 - = 30.0	87.5	2625)	6.4	41000
$\frac{6.6)}{2}$ = 43.2	87.5	3780)		
= 5.0	150.0	750)	2.84	5500
.67) = 7.9	150.0	1185)		
+ 10.04) = 154.2	120.0	18500)	8.25	180000
10.04) = 27.5	120.0	3300)		
	W_R	30140	7.53	226500

$$c.g.(1+2) = \frac{6x6.6x3.3+(2x8.4x(6.6+4.2))+(2x8.4(6.6+8.4/3))}{73.2} = 6.4\text{ ft.}$$

$$c.g.(3+4) = \frac{1.67x3(3+3.6/3)+3x1.5x1.67+1.79x3x1.5}{12.9} = 2.84\text{ "}$$

$$c.g.(5+6) = \frac{8.25x3x1.5+3x1.79x2x3/_{2x3}+10.04x12x9+6x11x5.67}{181.7} = 8.25\text{ "}$$

$$c.g.(1+2+3+4+5+6) = \frac{226500}{30140} = 7.53\text{ ft.}$$

Horizontal Forces

$P_1 = 0.17x120(15.71^2 - 10.04^2) = 2900$ #s.

$x_1 = c.g. = \frac{(15.71 - 10.04)^2}{6(15.71+10.04)} = .21$ ft. below the center of the surface "bc" or .21+.67 = .88 ft. below the surface of the water.

$P_2 = 0.17x120(10.04^2 - 8.25^2) = 755$#s.

$x_2 = c.g. = \frac{(10.04 - 8.25)^2}{6(10.04+8.25)} = .03$ ft. below the center of surface "ab" or 2.56 - .03 = 2.53 ft. above the water line.

$M_A = -P_2x2.53+P_1x0.88 = 3655X_3$

$X_3 = \frac{-1910+2550}{3655} = .18$ ft. above the water line.

Stresses

I_f - $N = 48,000$#s; $r_{ave} = \frac{N}{a} = 3200$#s/sq.ft; $e = 2.18$ ft.

$r_{max} = 3200(1+\frac{6x2.18}{15}) = 5950$#s/sq.ft.

$r_{min} = 3200(1 - .87) = 415$#s/ sq.ft.

I_b - $N = 50,900$#s; $r_{ave} = \frac{N}{a} = 3390$#s/sq.ft; $e = 1.63$ ft.

$r_{max} = 3390(1+\frac{6x1.63}{15}) = 5320$#s/sq.ft.

$r_{min} = 3390(1 - .57) = 1460$#s/sq.ft.

I_c - $N = 48,000$#s; $r_{ave} = \frac{N}{a} = 3200$#s/sq.ft; $e = 1.59$ ft.

$r_{max} = 3200(1+\frac{6x1.59}{15}) = 4980$#s/sq.ft.

$r_{min.} = 3200(1 - .56) = 1415$#s/sq.ft.

dent that the remaining stresses are not as great
, since "e", their eccentricity, is of a smaller

Condition of Loading II

Volume(ft^3)			Wt/ft^3	W(#)	c.g(ft)	M(ft#)
Same as Loading I				30140	7.53	226,500
1.04x15	=	15.6	120	1870	7.50	14,000
			W_R	32010	7.53	240,500

6+7) = $\frac{240500}{32010}$ = 7.53 ft.

s

x120$(16.75^2-11.08^2)$ = 3220#s.

= $\frac{(16.75 - 11.08)^2}{6(16.75+11.08)}$ = 0.19 ft. below the surface

or .19+.67 = 0.86 ft. below the water line

x120 $(11.08^2-9.29^2)$ = 735#s.

= $\frac{(11.08 - 9.29)^2}{6(11.08+ 9.29)}$ = 0.03 or 2.53 ft. above the surface of the water.

2.53+P_1x0.86 = 3955x3 = 0

$\frac{) + 2770}{3955}$ = 0.23 ft. above the surface of the water.

)250#s; $r_{ave} = \frac{N}{a} = \frac{50250}{15} = 3340$#s/sq.ft; e= 2.0 ft.

40 $(1+\frac{6x2.0}{15})$ = 6000#s/sq.ft.

40(1- .80) = 1670#s/sq.ft.

000#s; $r_{ave} = \frac{50000}{15} = 3330$#s/sq.ft; e = 1.4 ft.

30$(1+\frac{6x1.4}{15})$ = 5200#s/sq.ft.

30(1 - .56) = 1460#s/sq.ft.

Conditions of Loading III

l horizontal loads are the same as in conditions of

N $= 53000\#s;\ r_{ave} = \frac{53000}{15} = 3530\#s/_{sq.ft;}\ e = 1.50$ ft

x $= 3530(1 + \frac{6x1.5}{15}) = 5540\#s/_{sq.ft.}$

in $= 3530(1 - .60) = 1415\#s/_{sq.ft.}$

nce none of the stresses in the abutment exceed the

, the assumed design will be considered final.

APPROVAL

The foregoing thesis is hereby approved as a creditable study of an engineering subject, carried out and presented in a manner sufficiently satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is to be understood that by this approval the undersigned does not necessarily endorse or approve any statement made, opinions expressed, or conclusions drawn therein, but approves the thesis only for the purpose for which it is submitted.

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